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THE NEW ARMORED CRUISERS authorized by the last Congress will each have 23,000 HP. and a 21½-knot speed. By a vote of four to one the Naval Board of Construction has rejected Engineer-in-Chief Melville's proposition to provide for a 23-knot speed with 25,000 HP. The new boats will be 3½ knots slower than the four English armored cruisers of the "Drake" class, to offset which a more powerful battery is to be provided, namely four 10-in. guns in two turrets. Admiral Melville will present a minority report, taking the ground that the intent of Congress has been evaded by designing what are practically battleships instead of fast cruisers.

THE VIAUR VIADUCT, over the valley of that name and connecting by rail the towns of Carmausc and Rodez, in France, was officially opened to traffic on Oct. 5. This viaduct is a steel arch of 220 m., or 721.6 ft. central span, with a total length of 1,509 ft., including the two half-arches, or cantilevers, at the ends. The track is 393 ft. above the bed of the stream below. The viaduct cost about $600,000, and was built by the Batignolles Co., under the plans of M. Bodin, and directly under M. Volontat, as Chief Engineer. The viaduct was authorized in 1889, and has been under construction for some years; it was quite fully described in Engineering News of Sept. 6, 1900.

A CIVIL SERVICE EXAMINATION will be held at the Navy Yard, New York, Oct. 30, 1902, or as soon thereafter as practicable, for the purpose of establishing an eligible register for ship draftsman from which selections will be made to fill existing vacancies in the department of Construction and Repair, Navy Yard, New York, with pay as follows: First-class leading ship draftsman, $6 per diem; second-class leading ship draftsman, $5.52 per diem; first-class ship draftsman, $5.04 per diem; second-class ship draftsman, $4.48 per diem; third-class ship draftsman, $4 per diem; fourth-class ship draftsman, $3.60 per diem; first-class assistant ship draftsman, $3.28 per diem; second-class assistant ship draftsman, $2.80 per diem; third-class assistant ship draftsman, $2.40 per diem; fourth-class assistant ship draftsman, $2 per diem. The examination will be open to all comers who can give evidence of experience in the kind of work for which they seek employment, and who are citizens of the United States. Persons now holding positions at the yard will be admitted to competition on the same footing as other applicants. Applications will be addressed to the "Commandant, Navy Yard, New York," and must be delivered to him on or before Wednesday, Oct. 29, 1902. No application received after that date will be considered.

THE PRODUCTION OF RUSSIAN PETROLEUM in 1901 was 85,000,000 barrels as compared with 69,000,000 barrels in the United States. Russian coal is inferior and high priced, $7 a ton, so that the oil at 46 cts. a bbl. costs far less than coal for fuel, assuming 3½ barrels of oil equal to one ton of coal. Baku, on the Caspian Sea, is the main center of production, and the oil is transported through pipes 160 miles, than 400 miles by rail to the seaboard at Batum. Russia as yet exports less than half as much crude petroleum and only about one-quarter as much refined oil as does the United States.

FUEL BRIQUETTES, made from brown coal (a species of lignite), peat, and the dust and waste of coal mines, are extensively used in Germany, and their manufacture is said to be an important industry. In a report by Mr. Frank H. Mason, U. S. Consul-General at Berlin, it is stated that for use in locomotives and under steam boilers, and for heating in various processes of manufacture, they have three special advantages: 1, They are clean and convenient to handle; 2, They ignite easily and quickly and burn with a clear intense flame; 3, They are cheap and practically smokeless. About 90% of the product is controlled by a syndicate of 31 firms or companies. The output of this syndicate in 1901 was 1,566,385 tons; while the total output was about 1,643,000 tons. Of this total, 749,000 tons were taken by railways, 497,000 tons by factories and industrial work, 150,000 tons by German naval and merchant steamers, and 124,000 tons were sold to retailers. The average price was $3.16 per ton. The syndicate makes use in the production of these briquettes of a binder or matrix of mineral pitch, of which 117,000 tons were used, costing $10.25 per ton.

THE MOST SERIOUS RAILWAY ACCIDENT of the week was a butting collision between passenger trains on the Vandalia Line, near Reelsville, Ind., on Oct. 15. Two men were killed in the wreck, while several others were probably fatally hurt. The blame for the accident is placed on a telegraph operator, who, it is claimed, failed to deliver meeting orders to one of the trains.—An almost equally serious wreck occurred on Oct. 20 on the Calvert, Waco & Brazos Valley branch of the International & Great Northern R. R., near Calvert, Texas. Two men were killed and a dozen or more injured; the cause of the accident is not stated. A number of other train accidents are reported, several of them claiming one victim each.

A WOODEN WATER TANK failed at or near Lead, S. Dak., on Oct. 8. It is said to have been one of four belonging to the Homestake Mining Co., and is described as of red cedar, 50 ft. in diameter and 25 ft. high. The failure occurred just after the tank had been completed and filled, and is attributed to the snapping of one of the iron (or steel?) hoops.

THE RAILWAY WATER TANK of the New York, New Haven & Hartford Railroad Co., at Provincetown, Mass., which failed on Oct. 2, was built about eight years ago. It was of wood, 24 ft. in diameter and 16 ft. high, and besides ordinary railway service it supplied several tenement houses and a wharf. The exact cause of the failure is uncertain. The hoops and staves gave way, leaving the supports in perfect condition. Mr. G. T. Taylor, of Boston, Superintendent of the Plymouth Division of the New York, New Haven & Hartford Railroad Co., to whom we are indebted for the foregoing information, states that "one hoop was apparently defective where welded, but in the judgment of our experts the failure of one hoop should not have caused the tank to collapse."

FIRE PREVENTION in railway passenger cars has been made the subject of special study by the State Railway Department of Prussia, Germany. As a result of this study, it has been decided to test by trial in actual use the following improvements, among others, in passenger-car construction: Covering the floor planking with asbestos sheet and tin, with sleeves of similar material at all openings in the floor. Impregnating the woodwork of floor and walls after the Gautsch system. Protecting the seat upholstery by a covering of asbestos or similar material. Using woolen curtains instead of linen, and substituting less inflammable floor mats for those now in use.

FIRE PROTECTION AT THE ST. LOUIS EXPOSITION will include an extensive water-works system, said to be entirely independent from the city works, but with connections therewith for emergency use. High-pressure pumps connected with Arrowhead Lake, having a capacity stated as 8,000,000 gallons, will supply 13.5 miles of 24 to 4-in. cast-iron pipe, and 19.8 miles of 6 to ¾-in. wrought-iron pipe. Some of the pipe will run along the roofs of the buildings and there will be automatic sprinklers on the towers. Another special feature will be "deck-turret revolving stand-pipes," mounted on platforms 30 ft. high, placed within the exhibit buildings. These nozzles are designed to throw a 3-in. stream under 150 lbs. pressure. There will be 375 fire hydrants and 843 24 to 4-in. gate valves.

ELECTRICAL SUBWAYS for a portion of Brooklyn Borough are to be required in accordance with a resolution of the New York Board of Estimate and Apportionment instructing Mr. Robert Grier Monroe, Commissioner of Water Supply, Gas and Electricty, to notify owners of wires within the central district of the borough, to place the same underground within a time to be fixed by the commissioner. The only exceptions will be trolley contact wires and wires already "securely attached to the structure of elevated railways in accordance with the rules" of the department.

AN EGG-SHELL CAN WITHSTAND an internal pressure of 65 lbs. per sq. in., and an external pressure of 675 lbs. per sq. in.; according to experiments made by Mr. Albert E. Guy and published in the "American Machinist" for Aug. 18 last. The eggs were taken at random and the contents emptied by first boring a 1-16-in. hole at each end. The shells were tested in three ways—by pressure applied on the long axis; by internal pressure on the whole shell, and by external pressure. As the egg-shell is porous, the difficulties met with in trying the last two tests were overcome by using an ordinary toy balloon of thin rubber. For the internal test, a 1-16-in. tube, plugged at one end and with a hole drilled in the side, was inserted in the balloon. The balloon and tube were then put through holes carefully drilled in the ends and the balloon tightly tied outside the shell. The small tube was soldered to a ⅛-in. pipe plug for connection with the pump. For the external pressure test the egg-shell was tied up inside the balloon. Under end pressure the shells gave away at about 60 lbs. In the test for internal pressure, the gage showed from 32 to 65 lbs. per sq. in. The shell withstood from 425 to 675 lbs. per sq. in. applied externally to the egg-shell in a closed reservoir prepared for the purpose.

THE SEWERAGE SYSTEM OF LONDON, according to the recent annual address of Sir J. McDougall, Chairman of the London County Council, now serves an area of 140½ square miles and a population of 5,137,000. During the last fiscal year the average daily flow of sewage and drainage was 282,000,000 U. S. gallons and the sludge taken out to sea averaged 7,607 short tons per day, or 5,394 lbs. per 1,000,000 U. S. gallons. The net cost of maintaining the sewerage and sewage disposal works was about $1,200,000, which amounts to $11.66 per 1,000,000 gallons, or about 23 cts. per capita per year.

WATER PURIFICATION BY OZONIZED AIR after the Vosmaer-Lebret process is being worked at two places in Holland, Schiedam and Nieuwerslius, near Amsterdam. The coarse impurities in the water are removed by a rapid filter, and the water is then pumped into the top of the ozonization tank, whence it flows out at the bottom and into a storage tank. One of the two plants referred to is said to have a capacity of 5,000 to 8,000 gallons per hour, working a surface water of poor quality. The ozonized air, which is bubbled through the purification tank, is produced by silent discharges of electricity at 10,000 volts in a special apparatus. The current is supplied by an alternator at 110 volts, 100 p. p. s., and its pressure is raised to 10,000 volts by a transformer. Air is fed into the ozone generator at a rate of about 1½ cu. ft. per minute, and is then pumped to the purification tank. Previous articles on the treatment of water by ozone may be found in Engineering News of Oct. 19 and Nov. 23, 1899, and Feb. 8 and 22, 1900.

UNDERGROUND WATER RIGHTS affecting the water supply of Brooklyn, N. Y., continue to be passed upon in a common-sense manner by the appellate division of the New York State courts. Two opinions, covering four suits on appeal, have recently been filed. Each opinion affirms the right of the city (now Greater New York) to pump water from its driven wells, and, in effect, merely allows damages to market gardeners or farmers to the amount of the decreased rental value of their land.

THE NEW ORLEANS MERGER ACT, by means of which existing boards in charge of public improvements were consolidated, and a new and enlarged sewerage and water board created, has been declared legal by a lower court. It is said that the plaintiff taxpayers will appeal to the Supreme Court of Louisiana.

STEEL RAILS FOR AMERICAN RAILWAYS, ordered abroad by one New York firm, are said to aggregate 400,000 tons since February. Of this England furnishes 50,000 tons, Germany 300,000 and Belgium 150,000.

450-FT. STEEL ARCH BRIDGE OVER THE RIO GRANDE ON THE PACIFIC RY., COSTA RICA.

(With two-page plate.)

The principal railway line of Costa Rica is the Atlantic Ry., running from Port Limon, on the eastern seaboard, to San Jose, the capital of the republic. For some time construction has been in progress on an extension of this road from San Jose to the Pacific coast. This new line is known as the Pacific Ry., and of its total length of 60 miles about 46 miles are completed and 14 miles are approaching completion. Much benefit is expected to the agriculture and commerce of Costa Rica from this new railway, but its chief interest to engineers comes from the fact that its construction has necessitated the building of a steel arch bridge across the Rio Grande, which is probably the largest bridge in Central America, and one of the largest of its type in the world. Through the courtesy of the builders, Milliken Bros., 11 Broadway, New York city, we are able to give our readers a number of illustrations and a general description of this notable structure.

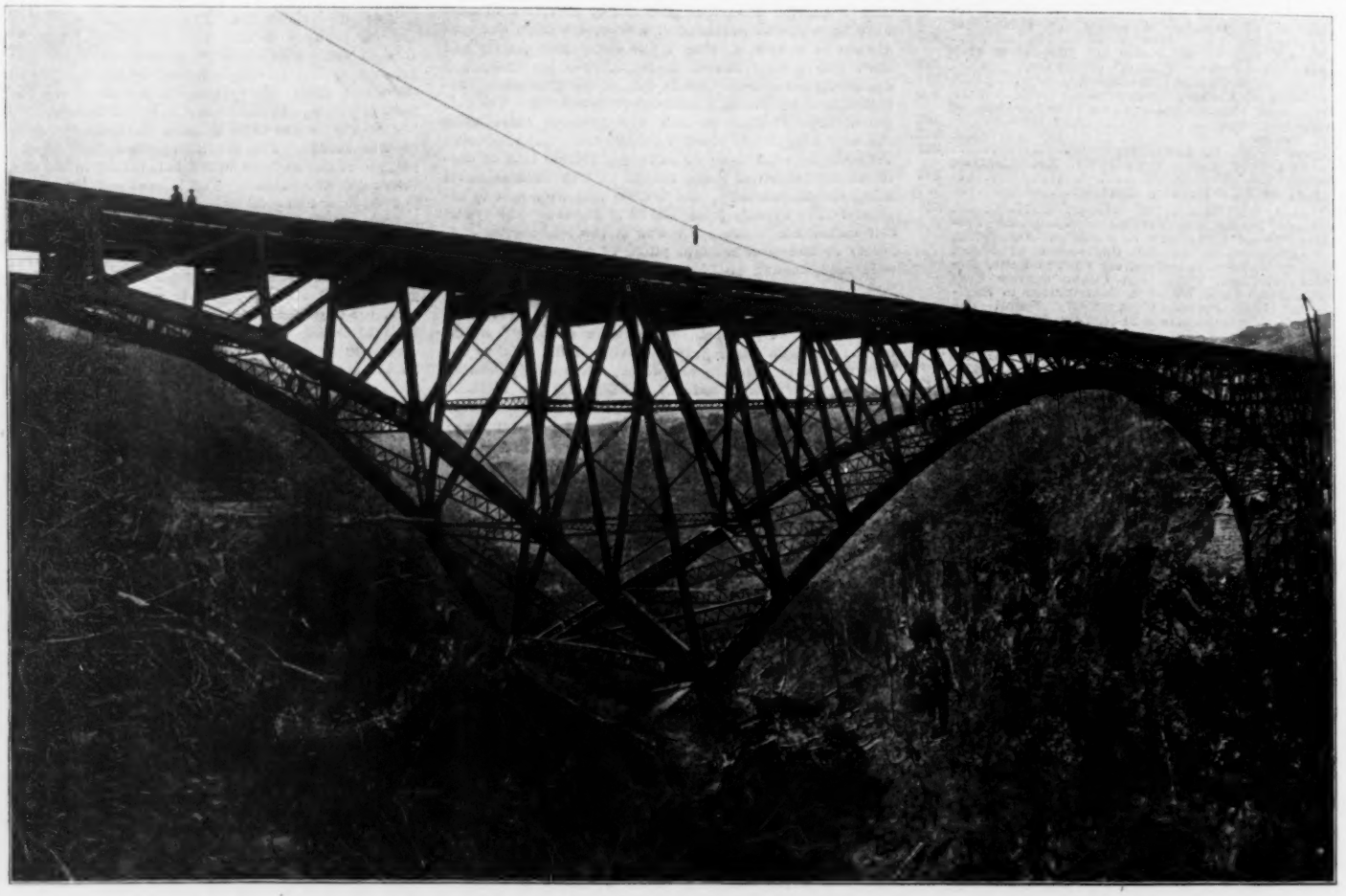

FIG. 1. GENERAL VIEW OF STEEL ARCH BRIDGE OVER THE RIO GRANDE; PACIFIC RY., COSTA RICA.
Theodore Cooper and Gunwald Aus, Ms. Am. Soc. C. E., Engineers; Milliken Bros., New York City, Builders.

GENERAL DESCRIPTION.—The Rio Grande bridge carries the Pacific Ry. over the river of that name at a point about 26 miles west of San Jose. At the point where the crossing is made the Rio Grande runs through a deep gorge with nearly vertical sides. To make the crossing a structure about 800 ft. long was required, with its rails some 340 ft. above the water. The structure designed for the crossing consisted of a braced arch of 448 ft. 8¼ ins. span across the river, and two shore arms, each 118 ft. 1½ ins. long, making a total length of steel structure of 684 ft. 7½ ins. between end pins. The bridge is for single track; of riveted construction, and carries the track on its top chord. A very good idea of the character and appearance of the structure and of the gorge which it spans is had from Fig. 1.

SUBSTRUCTURE.—The substructure of the Rio Grande bridge consists of two abutments and two pairs of piers, or skewbacks, to carry the arch trusses. The half profile, Fig. 2, shows one abutment and one pier. The abutments are of concrete masonry and have embedded in them the anchorages required for erection. Details of one of the anchorages is shown by Fig. 3. As will be seen, each anchorage consisted of eight 3⅛-in. anchor bolts connecting with an I-beam grillage at the bottom of the abutment and straddling the end floor beam with a yoke. The uplift on the shore arms amounted to about 280,000 lbs. for each truss and the abutments and anchors were designed to resist this load, but in order not to have too much pressure, and, consequently, great frictional resistance on the screw threads of the anchor bolts when lowering the arms into position after they met at the center, a considerable load was superimposed on the shore arms, so that no such strain ever came on the anchorage.

The skewbacks call for little description except to note that they were built so that the planes of their bearing faces were perpendicular to the planes of the trusses, and not, as is the more common practice, perpendicular to a vertical plane. The pressures which come on the skewbacks during and after erection are indicated on Fig. 2. The foundation bed of the skewback masonry was carried to solid rock.

SUPERSTRUCTURE.—The superstructure of the Rio Grande bridge has been previously described simply as a steel arch. As a matter of fact, and as is shown by the diagram, Fig. 4, the main span is a two-hinged braced arch, with two shore cantilever arms each 47 ft. 3 ins. long. Between the ends of the shore cantilevers and the abutments are two suspended spans, each 70 ft. 10½ ins. long. These suspended spans rest on the abutments at one end, and are carried by the cantilevers at the other ends, the connections being at U_3 and L_3, Fig. 4. The nature of the construction calls, of course, for slotted pin connections at U_3 and L_3. To permit the two halves of the main arch span to be erected as cantilevers from each shore, however, the joints U_3 and L_3 were spliced, the shore ends of the suspended spans were anchored down to the abutments, as previously described, and the top chords were designed to carry the entire tension from erection loads. After erection the splice plates were removed from the joint L_3. The upper chord joints U_3 were, however, left solidly riveted up, as it was not thought serious to allow a slight binding moment in the comparatively shallow chord under partial loading, considering that a much more rigid structure is obtained by not interrupting the top lateral system at this point. The object of introducing the suspended shore spans was to prevent the possibility of any hammering on the abutments. It will be understood, of course, that the abutment anchors were intended chiefly for erection purposes.

The bridge was designed under Cooper's specifications for a live load of 2,800 lbs., and a dead load of 3,000 lbs. per lineal foot of span. The top lateral system was proportioned to resist a stationary wind load of 150 lbs. per foot of bridge. The moving wind load was supposed to be transmitted through the sway bracing into the bottom lateral system, so that this system is proportioned for a stationary wind load of 150 lbs. per foot of bridge, and a moving wind load of 300 lbs. per foot of bridge.

The braced arch was originally figured in strict accordance with the method given in Prof. Greene's book on arches, the general formula first presented by Prof. Clerk Maxwell,

$$H = \frac{\Sigma\, S_0\, S_1 \frac{M}{A}}{\Sigma\, S^2 \frac{M}{A}},$$

being graphically calculated and summarized, as fully illustrated by Prof. Greene. This method was found to be exceedingly tedious for such a great number of panels, and as the bridge had to be refigured on account of a change of length, a

modified method was used, which, in the meantime, had been developed by Mr. Theodore Cooper, M. Am. Soc. C. E., who had been called in as consulting engineer, and, who, with Mr. Gunwald Aus, M. Am. Soc. C. E., made all the calculations. This modification, it is stated, "consists in developing the formula for H for two symmetrical panels and doing the summarizing in the formulas for two such points whereby a very simple formula is obtained." In regard to this formula the contractors, Milliken Bros., make the following statement:

The present article does not permit of a full discussion of the merits of this modified method; it will be discussed fully at another time. Suffice it to say that a very great amount of labor is saved and that the resulting formulas give considerable suggestion as to the relation of depth of span to length of span for economical design, also that the resulting formulas show a certain progressive increase of factors, so that the formulas may at once be written for any span or panel number after once being developed for a given span or panel number.

The temperature stresses were calculated for a total variation of temperature of only 70°, as the bridge is built in a country where no great variations occur. As will be seen from the strain sheet, Fig. 4, they form, even for this small variation of temperature, a very considerable percentage of the total stresses.

In the design of the arch there were two or three rather notable changes from ordinary practice. In the first place, the shoes carrying the main span were constructed entirely of rolled steel, so that no cast steel or cast iron were used in the bridge. Fig. 5 shows the construction of the shoes in detail, and Fig. 6 is a view of two of them in the shops of Milliken Bros., just before shipment. Another innovation was the setting of these shoes in the same plane with the trusses, instead of in the vertical plane, as is usually done. Stated another way, the skewbacks were set at an angle with the vertical so that the shoes were in a plane with the trusses, and the pins were at right angles with the trusses and with the axes of the shoes. The character of the structural details of the trusses is indicated by Fig. 7, which is a detail of the second post from the left skewback.

ERECTION. — Briefly described, the method of erection was as follows: The shore spans, comprising the 47-ft. cantilever and 70-ft. suspended span on each side, were erected on falseworks. The connections between the suspended spans and the cantilevers were then spliced, as before described, and the shore ends of the suspended spans were anchored to the abutments. The arms of the arch span were then built out from each skewback as cantilevers without falseworks until they met, and were connected up at the center. During erection the top chord was, of course, under tensile strain, and after the center connections had been made this strain was relieved by slacking the anchor bolts. The arch came together almost perfectly, and no changes had to be made in any member in the field to fit it to its proper position. To have secured these perfect results reflects much credit on the shop and field work of the builders.

Particular care and attention were given to laying out the base lines at the bridge site, and to locating the exact points for setting the shoes. Two steel tapes were purchased in the United States, and were sent to Washington and standardized by the United States Government, and were corrected for every 10° of variation in temperature, and were furnished with spring tension arrangements so that, under a given load the tape gave exact measurements. One of these tapes was sent to Costa Rica to be used as their standard in laying out the work; the other tape was retained in the United States, and was used in the shops as a standard. The work of erection in the field was carried on under great difficulty. All of the work for half of the bridge had to be transported across by the cableway, which is shown by Fig. 8. The crudest kind of falsework was used to save the expense of sending down much lumber for the erection of the shore spans.

In order to allow a certain amount of leeway, to make up for any possible small errors in the placing of the shoes, the center sections were made somewhat shorter than the calculated length, the holes in one side of the center gussets and splice plates left blank to be drilled in the field after all parts had come to a full bearing, and a number of planed shimming plates were shipped along to be used as needed. As a further safeguard, the center chord sections were held at the shop until the arms came sufficiently close together to allow of exact measurements, so that any errors could be corrected in the shop before shipment. It was, however, found that no corrections of any kind were necessary, and the works were telegraphed to ship the center sections exactly as turned, showing the great accuracy obtained in the shop and in the field relative to this work.

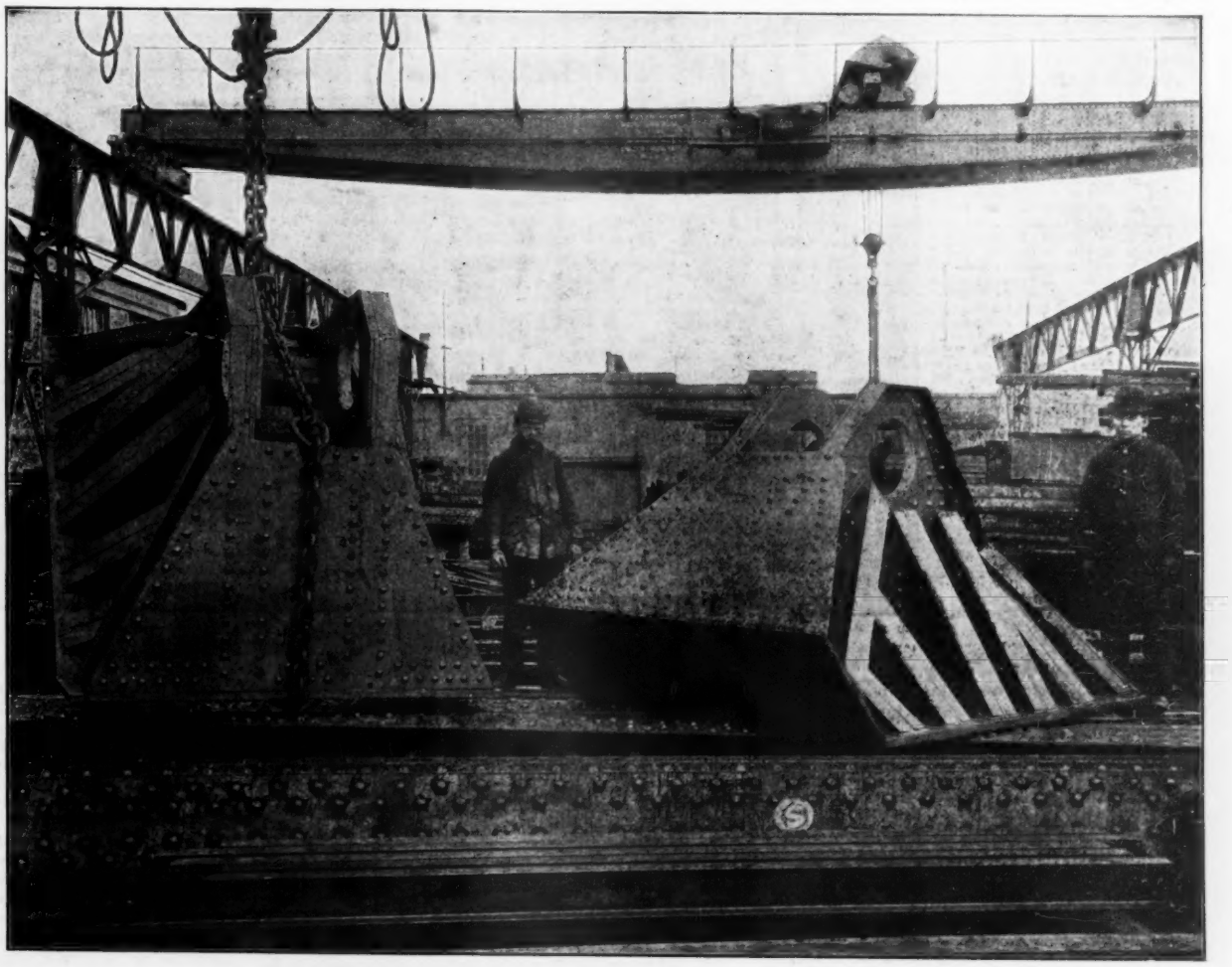

FIG. 6. VIEW OF ARCH SHOES AT SHOP READY FOR SHIPMENT.

ANNUAL MEETING OF THE AMERICAN GAS LIGHT ASSOCIATION.

The thirtieth annual meeting of the association was held in New York city on Oct. 15 to 17, inclusive, the last day being devoted to visits to the gas plants of the city and other points of interest.

THE PRESIDENT'S ADDRESS.

Mr. Wm. R. Beal, the president of the association (also president of the Central Union Gas Co., of New York city), opened his address by stating that of seven annual meetings held in New York this was the first one to find the gas companies of the city free from ruinous competition or other troubles. Beginning in 1823, there have been 25 different gas companies within the present limits of Greater New York. The few companies remaining are in good condition.

Mr. Beal expressed himself as strongly in favor of inclined retorts, which, with coke ovens, mean permanent prosperity for gas companies. In the smaller cities he thought that all public utilities not under municipal ownership should be controlled by one corporation, and he also favored merging the interests of gas companies in different localities. Municipal ownership he thought desirable in an ideal community, but not under present conditions. Officials of gas companies should study the wants and interests of the public as a matter of good citizenship, and should stand ready to coöperate with the municipality. Conservative gas managers are looking forward to State Gas Commissions (like the one in Massachusetts.—Ed.), and to the time when the earnings of gas companies will be kept down to returns on capital similar to those in other lines of business. A committee to study the movement for public ownership, and another one to prepare arguments to influence the public in favor of measures against electrolysis was recommended.

UNIFORM GAS ACCOUNTING.

Mr. Alex. C. Humphreys, the recently installed President of Stevens Institute of Technology, Hoboken, N. J., submitted a report for the committee on this subject appointed in 1900. The committee, which includes some of the leading gas engineers

and accountants of the country, presented an exhaustive system of gas accounting, with a longer and a shorter form. These schedules, in their earlier form, were sent to all the members for criticism, and revised accordingly. The criticisms were few and far between, so the committee was authorized to again send out the forms, and to include such suggestions as it thinks proper up to Nov. 15, 1902. With this understanding, the report and system of accounting were adopted.

REPORT OF RESEARCH COMMITTEE.

This was submitted by Mr. Alten S. Miller. The committee has gone on with its work on standards for pipe specials, designed to reduce the number of patterns to a minimum. Cuts of these standards will be sent to the members for criticism, and the standards will be submitted to the next convention. It was thought that pressure enough can be brought to bear upon the foundries to secure the adoption of the standards.

NOTES ON CEMENT MASONRY.

Mr. I. N. Knapp, of Philadelphia, reviewed this subject in detail, quoting largely from other writers. The subject of testing cement was introduced as follows:

I cannot go fully into the question of testing hydraulic cements in the limits of this paper—it is an arbitrary matter and chiefly valuable as a means of comparison—but will point to a few facts brought prominently out by my own experience. What it is necessary to know about any cement we propose to use is that it will set and harden into a solid mass sufficiently strong for our purpose when properly mixed with sand and water, and make a mortar for brick, stone or concrete that will firmly adhere to any surface with which it may be in contact, and that it will endure without change of form or loss of strength for a long time. For these properties the gas superintendent or manager had better depend upon the reliability of an established brand of cement, with occasional samples sent to a testing laboratory, rather than to rely on his own tests, unless he has time to learn the business of testing cements, or his construction work is extensive enough to justify having an assistant for such work. We are certainly pretty safe in accepting for gas-holder tanks, engine foundations and other important work a brand of cement which has been in actual use in such class of structures for a period of years and given satisfaction.

A number of tables and diagrams, by the author and others, were presented to illustrate the bad results due to an excess of water, the folly of regaging cement, and the like.

The author thus described his method of setting large dimension stones:

In setting large dimension stones such as engine beds, pier copings, column base stones and the like, I allow for a 1-in. joint to bed the stone. I then set the stone on hard wood wedges and clamp it in place by the foundation bolts. I then take Portland cement with an equal portion of coarse sand and just dampen the mixture. I tamp this mortar in the joint under the stone with a hardwood tamper or a steel bar. In this way a large stone can be set more quickly and accurately, and the bedding is stronger than by lowering the stone into a bed of ordinary cement mortar; besides, the work can be done with ordinary labor. Engines, pillow blocks, floor stands and all heavy machinery set on a stone or brick foundation can be securely bedded down with cement mortar in the same way. The machine can be accurately set on hardwood wedges and bolted in place, and the mortar tamped in to the joint. Water should only show slightly when the mortar is thoroughly compacted in place.

Several members said that brands of cement with the best of reputation had been poor of late, perhaps partly due to the great demand for cement, and that safety demands that all cement be tested. Some one ventured to speak against concrete mixing machines, and thus raised a small army of men in their defense.

GAS RATES AND A FRANCHISE TAX.

Mr. Geo. McLean, of Dubuque, Ia., opened his paper on this subject with the following statement regarding the regulation of rates in Iowa, with particular reference to its bearing upon his own company:

Taxation of gas companies and public control of rates are related questions under the laws of Iowa; for the reason that the laws of our State provide for the taxation of the franchises of public service corporations, and provide, also, for public control of the rates of some of them. Under our Iowa statutes, the city council is empowered to regulate the rates of gas and electric light companies as frequently as it chooses, and it may not contract away this right. It may contract to pay a certain price for public service for a number of years, a lower court has held, and it may prescribe the maximum charge for domestic service. But it may not guarantee that this maximum charge will not be altered during the life of the charter. The obvious purpose of this statute is to protect the public against the incapacity or dishonesty of its own agents; against contracts authorizing extortionate charges for domestic service. The objection made is that it exposes lighting companies to blackmail and to the attacks of the malicious and prejudiced. Our own experience is that it does not increase the danger from these sources by comparison with charters providing for readjustment of rates at certain fixed intervals. Under the Iowa law, the life of the charter is limited to twenty-five years. If we could have a law fixing rates for the entire period of the charter, we might prefer that as offering an absolute guarantee against persecution in the form of attacks on rates. However, public sentiment is such that the choice seems to lie between a statute empowering the council to readjust rates for domestic service at any time, and a contract providing for readjustment at intervals not exceeding five or ten years. We prefer the Iowa statute because, in practice thus far, it has meant no regulation of rates except by competition. When the contract provides for readjustment of rates every five or ten years, the company is forced into politics periodically to protect itself against possible injustice. During the existence of the Iowa law, but one attempt has been made to regulate our rates. It was not made in response to public demand, but to gratify malice. We protested that the power to regulate rates is a reserved power, granted to the city council for the purpose of protecting the public against monopoly, that competition with other illuminating and heating agents is the best regulator of rates, and that where such competition was present, as in the lighting business in Dubuque, arbitrary regulations could not be justified. The justice of our protest and the absence of public demand for the exercise of the council's power were so apparent that the author of the proposition to regulate our rates by ordinance withdrew it. And while the statute has given us no trouble, except in this single instance, when the attack was so easily repulsed, it has given us an argument against the raiding company seeking admission to our field. It has enabled us to answer, if asked why a second company should not be admitted to our field, that if our rates are reasonable the public interest does not require the chartering of a second company, and that if our rates are unreasonable then, also, the public interest does not require the chartering of a second company; because the remedy lies not in duplicating the service, an expensive process for the public in the end, but in reducing the rates of the established concern to a reasonable basis, which the council in Dubuque has power to do. The statute affords us no guarantee that the council will not charter a second company, and, as a matter of fact, competing companies have been chartered in a few cities of Iowa since this statute was adopted; but it supplies a reason why the service should not be duplicated, and appeases public sentiment in favor of duplicating it, thereby increasing the liberty of the council to deal with the question on its merits. Those who protest against public control of the price of any commodity, denouncing the principle as socialistic and a menace to vested rights, should bear in mind that it is a condition and not a theory that confronts us. A considerable portion of public sentiment in many places demands public regulation, actual or potential, of the charges of public service corporations. The condition of this public sentiment is such that the issue lies between the actual exercise of this constitutional power of public regulation of rates at certain brief intervals and the reserved power to regulate at any time. I prefer to concede the reserved power to regulate at any time; because this means that the power will be exercised at less frequent intervals, if at all.

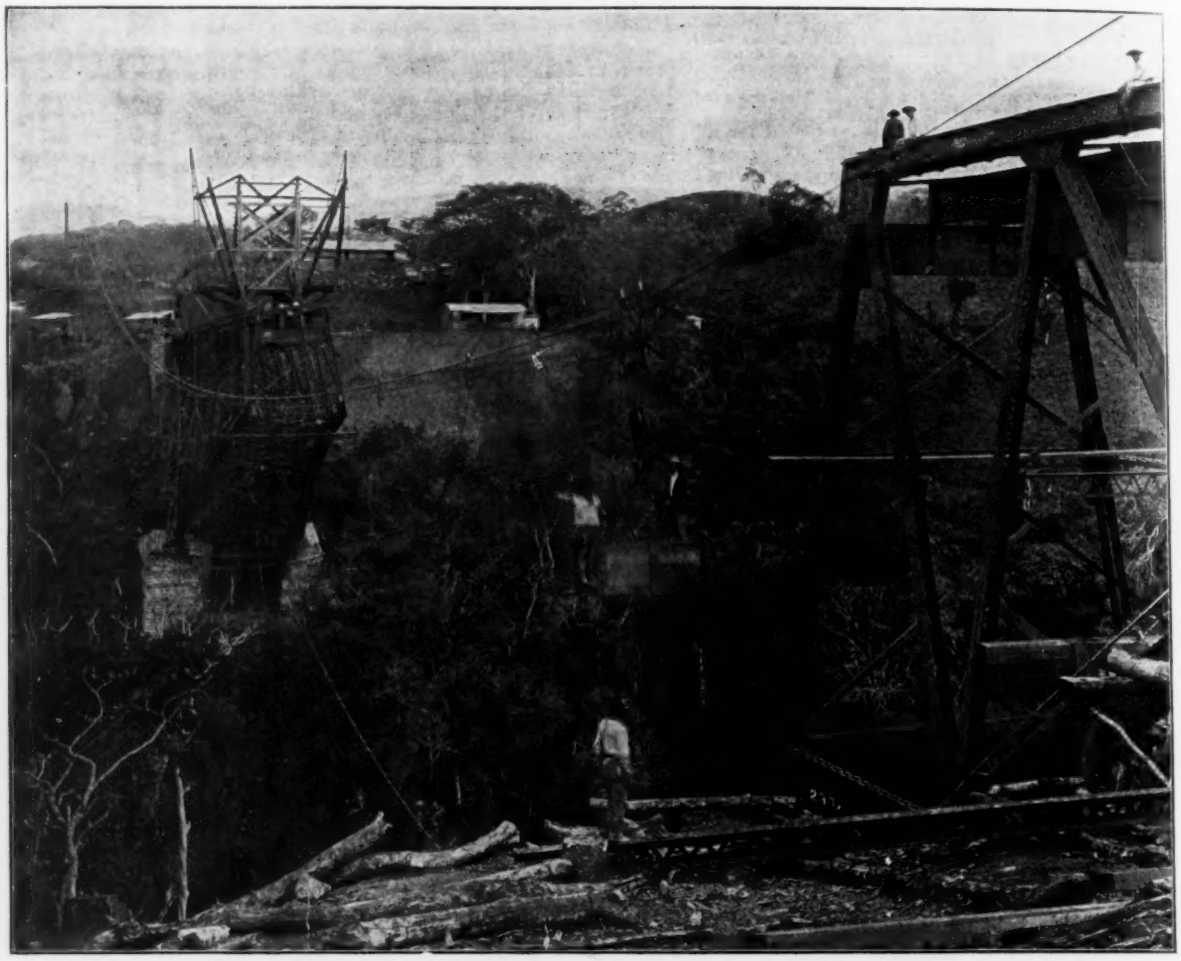

FIG. 8. VIEW SHOWING METHOD OF ERECTION.

On the subject of taxation, the author was not so well in accord with the views of disinterested students of the problem, for he failed to recognize the difference between a public service corporation and men or companies engaged in purely private business.

Mr. Alfred E. Forstall, Secretary of the association, said he thought the gas companies had been

playing ostrich long enough, and should now take a hand in working out a system of rate regulation and taxation which would be fair to everybody. President Beal said he believed in monopoly in the gas business, but it should be a regulated monopoly. He has favored State gas commissions for twenty years. Over-capitalization is harmful to the companies and the public alike.

OTHER PAPERS AND BUSINESS.

A paper on "The Care and Maintenance of Gas Burners" was sent in by Mr. MacDougald Dexter, of Columbus, Ga. He advocated regular inspection and repair by the gas company. At Columbus, weekly inspections, with renewals and adjustments as needed, is the practice. Clean chimneys are supplied in place of those badly soiled. Gas arcs were considered preferable to separate burners, because they require fewer mantles, "since one with a small break is still serviceable and has not to be renewed on account of danger to a chimney or small globe." In addition, the glassware, not being highly heated, is easily kept clean, and a man inspects four or more burners at one adjustment of his ladder.

There was a long paper, and a longer discussion, on "Why Milwaukee Did Not Build Inclines" (inclined retorts), the author of the paper being Mr. R. B. Brown, Engineer of the Milwaukee Gas Light Co. In brief, the reason given, which was supported by numerous estimates, was because vertical eights ("a bench setting two retorts wide and four high") were considered cheaper and to have a variety of structural and operating features in their favor. The lengthy discussion brought out the fact that while inclined retorts have been extensively adopted in England, and are frequently found in Continental Europe, they have been used but little in this country. Moreover, the comparative data regarding inclined retorts, on the one hand, and horizontal and vertical on the other, was said by several speakers to be both meager and partisan. It has already been noted that the president, in his annual address, spoke strongly in favor of inclined retorts, and it may be added that he also said that they are the chief improvement in carbonizing coal since the beginning of the gas industry.

A paper by Mr. Geo. S. Clarke, of Kansas City, Mo., on "The Cleaning and Repairing of Gas Stoves," described the thorough and systematic methods to that end employed in keeping up the 16,000 stoves loaned to consumers in Kansas City. The author was not present, and the reader of the paper gave some supplementary figures showing that repairs were made on 2,400 stoves in 1900 at a shop cost of about $2 each. Other items, chiefly small repairs on stoves in place, brought the total repair bill up to such a figure as to give an average of 43 cts. each for the whole 16,000 stoves in use. When to this is added capital charges, the speaker said, the moral to be drawn from free gas stoves is obvious.

The committee on gas advertising, or the representative from this association on the joint committee of the several societies, reported that the several gas companies had been asked to contribute to a general advertising fund at the rate of $2 per 1,000 population of the cities represented. Only one company had flatly refused to contribute, but about 20% of the respondents had conditioned their contributions on fairly complete coöperation of all the companies. A prominent publishing house is about to launch a gas journal for circulation among actual or possible gas consumers, at a rate of about 50 cts. per year. Apparently, the subscriptions are to be paid by gas companies, for it was stated that when once well introduced in homes it was expected that bona fide subscriptions would follow.

For the ensuing year the president will be Mr. Alten S. Miller, Chief Engineer of the New Amsterdam Gas Co., of New York city. Mr. Alfred E. Forstall, of Montclair, N. J., was reëlected Secretary and Treasurer. The next annual meeting will be held at Detroit.

MARRAM GRASS AS A SAND-BINDER, IN VICTORIA, AUSTRALIA.

In our issue of April 24 last will be found a query as to the action of certain grasses in binding sand together, and such information upon that head as we could then gather together. This material has brought to us a letter from Mr. H. V. Champion, M. C. E. and Assoc. M. Inst. C. E., of Melbourne, Australia, inclosing a paper read by him upon this subject before the Victorian Institute of Engineers. As literature upon the use of grasses for this purpose is scarce, a full abstract of the experience of Mr. Champion follows.

The author of the paper had been called upon to construct a highway between Williamstown and Melbourne, on a route where a previous road had been covered by blown sand for a length of about 2,000 ft., and in places buried to a depth of 3 or 4 ft. This sand came from pits dug to supply building sand to Melbourne, and prevailing northerly winds had thus converted about 300 acres of land into a desolate waste. It became necessary to devise some means for protecting the new road from this inroad of sand, and a previous trial of covering the sand with a thin coat of silt had utterly failed, owing to the fact that this surface was easily broken through by the carts engaged in the sand traffic; the silt covering was also expensive, costing about £27 per acre.

In older countries it has long been recognized that the most effective way of preventing inroads of blown sand is to create barriers of certain plants that flourish in sand. The surface is thus held in check and gradually prepared for greater stability and fertility. Along the Bay of Biscay the Pinus Maritima acts as an admirable barrier; Lathyrus Maritimus, or Sea Pea, is also thus used, with its exceeding long roots which multiply indefinitely; and in Victoria, the Arando Arenaria, or Marram Grass, had already been used with much success at Fort Fairy, and miles of sand hills reclaimed. At this place, where a few years ago there was not a sign of vegetation, there is now a succulent grass growing to a height of 4 ft. This Marram Grass is eagerly devoured by cattle, and is practically indestructible; burning, cutting or grazing only makes it thrive. This grass could be supplied at the actual cost of digging, packing and transport, and could be landed at Williamstown for £2 5s., or about $10.88, per ton.

At the Marram Grass plantations, at Port Fairy, it was found that parcels of the plant had originally been brought from India, and then planted in rows 6 ft. apart on lines at right angles to the direction of the prevailing winds; each plant was placed 2 ft. from its neighbor in the same row. It is roughly estimated that since 1888, 3,000 acres have thus been reclaimed and converted into profitable grazing land. In one case a road-cutting, 12 ft. deep, through pure sand, had been perfectly maintained by sodding the slopes with ordinary grass and then planting the immediate vicinity with the Marram Grass. At another point where the sea had invaded a piece of ground upon which this grass had been firmly established, the vertical face made showed that the roots of the Marram Grass presented a dense network from top to bottom of this 8-ft. face.

At Melbourne the following method of planting was adopted: An area of 60 acres was surrounded by a wire fence to keep off the cattle for the first two years. At first, 40 tons of the plants were delivered in bags, and the planting began. A foreman marked out the lines in the sand and 14 men did the planting. Holes were dug by one man while a follower planted and stamped in the filling, being careful not to injure the grass. In planting, all sandfilling or accumulation above the natural surface was removed, and the planting was done towards the sea-shore, or the source of the evil. If it were found that the drift was coming in the direction of the lines of grass, breaks, of not less than four rows, were planted at right angles to the drift. The actual planting of about 27 acres, by day labor at 6s. per day, cost £84 13s. 8d. And the cost per acre of thus reclaiming 27 acres of land, exclusive of fencing, amounted to £6 14s. 7d., or say $31.54.

For the first two years every precaution must be taken to see that the grass is not interfered with in any way. New shoots will then appear between the rows, and this new grass may be transplanted by cutting with a sharp spade, about 6 ins. below the surface and under the knots, from which the roots spring. After this transplanting the original grass is allowed to grow and to uniformly cover the surface. When the plants have arrived at maturity—say in three years—the fences may be removed and the land converted into grazing ground for cattle; this latter use is beneficial, as the manure deposited contains the seeds of other grass plants which quickly take root and thrive in the fertilized soil.

Experience in South Australia shows that this Marram Grass will flourish in inland districts with a very small rainfall.

THE ADOPTION OF A SINGLE GRADE OF STEEL IN STRUCTURAL SPECIFICATIONS.

By Albert J. Himes,* M. Am. Soc. C. E.

The proposition to adopt a single grade of steel for all ordinary uses to which structural steel may be put, has been frequently discussed of late, but the subject is of such importance that further remarks thereon would seem to be in order.

In the design of a metallic structure it is necessary to use a factor of safety large enough to cover all possible unknown conditions and insure that no part shall be strained sufficiently to cause rupture. It should be understood that, in using this factor of safety, it is not expected the structure will have a strength a certain number of times greater than it is proposed to use, but that its strength will be such that, notwithstanding numerous and various uncertainties and imperfections, no part of the structure will be strained to the limit of its strength. The factor of safety is then a function of the uncertainties and imperfections in the makeup of a structure, and should vary with their number and magnitude.

When steel first became a structural material, the most common objection to its use was its variable and uncertain quality as compared with wrought iron, and even now its behavior is at times very puzzling. But the quality of structural steel has been so greatly improved that its use has become universal, and this improvement has been largely due to the efforts of those engineers who have been foremost in its use and in the endeavor to improve its quality.

It is generally known that portions of the material represented by a single test as usually made at the mills, may vary in strength several thousand pounds from the result shown by the test. Three thousand pounds is a conservative estimate of such variation. It may, therefore, happen that some portions of material rolled for a specification of 52,000 lbs. to 62,000 lbs. may have an ultimate strength as low as 49,000 lbs. or as high as 65,000 lbs., and that some portions of material rolled for a specification of 60,000 lbs. to 70,000 lbs. may have an ultimate strength as low as 57,000 lbs., or as high as 73,000 lbs. When it is said that medium steel is injured by punching and the injured material must be removed by reaming, we may well ask what has happened to the so-called soft steel which may have a strength above 62,000 lbs., and is punched without reaming, because it happened to be represented by a test that pulled less than 62,000 lbs.

These considerations suggest some of the possible uncertainties which determine the factor of safety.

Uniform quality is most to be desired in structural material, and nothing has done more to secure that uniformity than the general use of the specification which permits the overlapping of the grades of soft and medium steel, so that material represented by a test which pulls between 60,000 lbs. and 62,000 lbs. may be accepted for either grade. It has been the steel-maker's ambition to produce a material which will pull between these limits, and the degree of his success is shown by the proposition that came before the recent convention of the American Railway Engineering and Maintenance-of-Way Association, to endorse a specification leaving a gap between the upper limit of soft steel and the lower limit of medium steel, so as to compel the manufacture of two distinct grades.

It is well known that a large proportion of the structural steel now being rolled satisfies the re-

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quirements of either soft or medium steel, but engineers who advocate such a proposition appear to be guided by the precise limits of a mathematical formula rather than by an intimate knowledge of the varying conditions which surround the manufacture of steel.

Only a few years ago specifications frequently provided for four or five different grades of steel to be used in a single structure with different unit stresses to be used for each grade. This method has fallen into disuse because of the difficulty of making sure that each member of the structure was made of the proper grade. In the case of such a structure, the factor of safety should provide for the possibility that a member might be built of a lower grade than was intended and should, therefore, be larger than where such a possibility did not exist.

To state the case differently, the specification implies a nicety of calculation and manufacture that cannot exist, and is, therefore, misleading.

By using a single grade of steel there is much less chance for any member to be made of a material having a different strength from that intended.

If we consider the conditions at the mills we may find a further reason why the general use of a single grade of steel may tend to a greater uniformity of product. It must be borne in mind that the material brought to the rolling mill has been ordered to suit the demands of its customers and that while its analysis is known, the heat treatment at the rolling mill may materially affect its final quality. Therefore, when the mills begin rolling, the question is frequently raised whether a certain stock will roll to a certain specification. The question is not always answered correctly, and sometimes material that should be medium proves to be soft, and vice versa.

It is true that if the maker would aim at a strength midway between the limits instead of a strength that will suit either grade, the product would be more likely to fulfil the specifications; but to think that no miscalculation or confusion can occur, that every slab is ordered for a specific use, and that its grade is fully and accurately determined, is a very great mistake. Steel-makers are human, and their machinery is not all clock-work. There are interruptions, changes, breakdowns, and all manner of annoyances to keep the mill superintendent studying how he can keep his mill running.

Now, if a very large percentage of his product is to have the same quality, it goes without saying that the quality will be more uniform, and the steel will cost less money than the same total weight divided among a dozen specifications.

Greater uniformity in quality permits the use of a smaller factor of safety; or, if the stresses remain the same, the factor of safety increases. Greater uniformity in demand means a greater uniformity in quality and a lessened cost of production.

Those who have observed the great number of cases in which tests fall between 60,000 lbs. and 62,000 lbs., will have no doubt of the ability of the mills to keep the tensile strength of their product between the limts of 56,000 lbs. and 64,000 lbs. Where special requirements make an unusual grade of steel desirable, those who have discussed the subject agree that a single standard should not be adhered to.

As to the treatment of the steel in the shops, first-class workmanship requires that all holes shall be reamed. There is no other way by which rivets can be made to do the work for which they are calculated. The case is like the uniformity of steel, the improvement in the quality of riveting tending toward a reduction in the factor of safety. If they who have been using soft steel without reaming object to the cost of the latter, it might be suggested that they omit a sufficient number of rivets to equal the difference in cost and drive the remainder as perfectly as possible. The result would be not inferior to their present practice.

In conclusion, it appears that for all ordinary construction the use of a single grade of steel having a maximum range in tensile strength of 8,000 lbs., and the reaming of all rivet holes, would materially lessen the variation of the actual from the calculated strength of the different parts of a finished structure, and so permit a more precise determination of its capacity.

OBSERVATIONS ON THE WEAR OF STREET RAILWAY RAILS IN SHEFFIELD, ENGLAND.

The wear of street railway rails at Sheffield, England, was discussed by Mr. C. F. Wike, M. Inst. C. E., City Surveyor of Sheffield, in a paper read before the recent Bristol meeting of the Association of Municipal and County Engineers and Surveyors.

Up to March 25, 1902, the municipal railway system included 50 miles, measured as single track. A total of about 160 cars were in use, of which one-third are single-deckers, weighing about 18,000 lbs., and the balance double-deckers, weighing some 19,000 lbs. The maximum grade is about 10½%, and the sharpest curve has a radius of 40 ft. Grooved girder rails are used, 108 lbs. in weight, 30 ft. in length, with 7½-in. flange, 7-in. total depth, 1 11-16-in. tread and a groove 15-16-in. deep. This gives a clearance of 7-16-in. between the wheel flange and the bottom of the groove. The specifications limit the carbon in the rails to 0.35 to 0.45%; manganese, 0.70 to 1.10; silicon, not over 0.1; sulphur, arsenic and copper, not over 0.8% each; and no other material except iron is permitted.

Mr. Wike's table of observations on wear, condensed by the omission of some records for a number of the observing stations, but retaining the most significant figures, is reproduced herewith. Mr. Wike comments on the table as follows:

There is a clearance of 7-16-in. between the [illegible] of the wheel flange and the bottom of the groove, and it is considered that when the rails have worn sufficiently to enable the flange to touch the bottom of the groove their life will be at an end. Before this can happen a sectional area of 0.706-in. of steel must be worn away. Observations have from time to time been taken to ascertain the rate of wear upon the rails, and the results are given in the table. These are interesting, as showing how much more the wear is regulated by the [illegible] than by the number of journeys, the reason for this being that the action of the brakes is far more destructive than the up-hill traction.

The paper also contains observations on the wear of points and crossings, and in the original paper the tables for these and for the rail records are supplemented by diagrams. The full paper was printed in the "Contract Journal" for July 16, and in the "Surveyor" for July 18, each of which is published in London.

Observations on the Wear of Street Railway Rails in Sheffield, England.

	Track.	Gradient.	Months wear.	No. of journeys.	Reduction, depth of groove, in.	Total possible reduction, sectional area of tread of rail, %
A...	down	1 in 15	34	764,200	0.25	54.1
	up...				0.09	15.1
B...	down	1 in 23	34	865,507	0.15	30.6
	up...				0.09	15.1
F...	down	1 in 787	34	593,704	0.125	20
	up...				0.06	8
K...	down	1 in 268	19	254,412	0.07	11.6
	up...				0.035	5
L...	down	1 in 15	34	166,734	0.07	11.6
	up...				0.03	3.9
N...	down	1 in 84	30	65,384	0.06	8
	up...				0.06	8
O...	down	1 in 79	26	172,558	0.06	8
	up...				0.028	3
P...	down	1 in 17	30	65,384	0.06	8
	up...				0.025	2.9

FAST EXPRESS LOCOMOTIVE FOR THE BIG FOUR R. R

We illustrate herewith a very heavy locomotive recently completed at the Brooks Works of the American Locomotive Co. for the Cleveland, Cincinnati, Chicago & St. Louis R. R. It is of the type which is fast becoming standard for heavy passenger service, having the firebox over the frames and the rear end carried on a pair of trailing wheels. The boiler is enormous, being 6½ ft. in diameter, and having a total heating surface of 3,340 sq. ft. The large diameter of the boiler, and of the driving wheels as well, makes it necessary to place the boiler very high, the distance from the boiler center to the rail being 9 ft. 8 ins. The boiler is designed to carry 200 lbs. pressure, and is made up with 13-16-in. plates in the largest ring. The cylinders are 20½ × 26 ins., and the maximum tractive effort at starting is 23,800 lbs., or 23.8% of the total weight on the drivers (100,000 lbs.). While this total weight gives nominal maximum loads per wheel of 25,000 lbs., the very high center of gravity of the locomotive must cause a considerable excess over this at times, for example when moving at slow speed around curves with considerable superelevation. The stresses which such large machines impose on the track and the resulting increased cost of maintenance are features which all engineers familiar

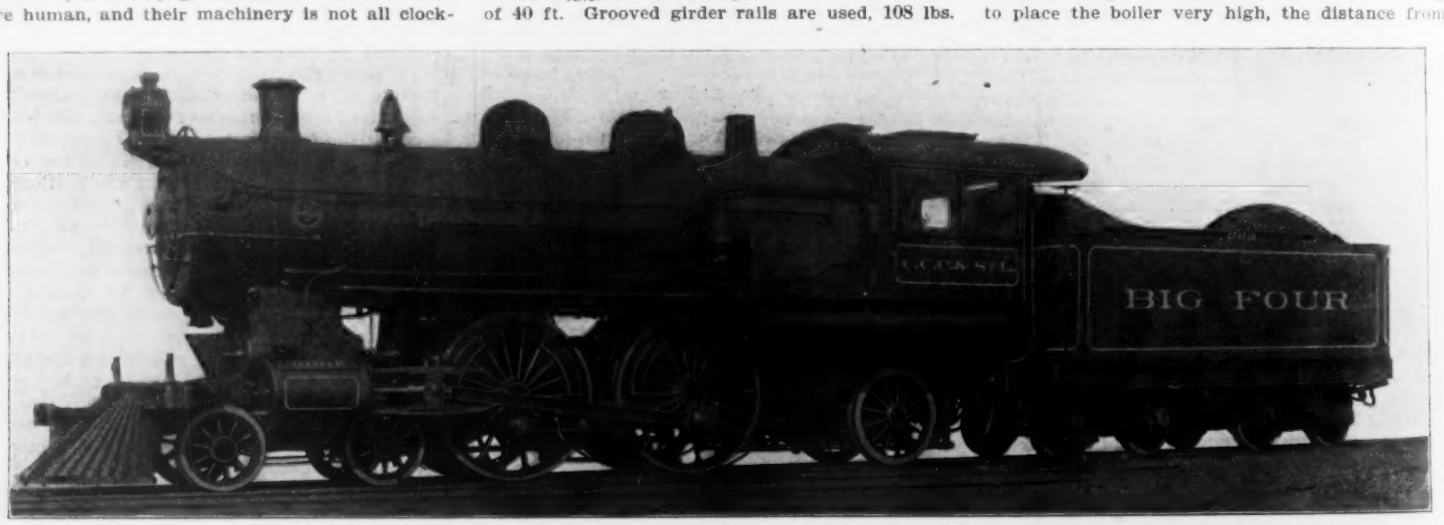

FIG. 1. HEAVY EXPRESS LOCOMOTIVE FOR THE CLEVELAND, CINCINNATI, CHICAGO & ST. LOUIS R. R.
W. Garstang, Superintendent of Motive Power; American Locomotive Co. (Brooks Works), Builders.

with present day maintenance-of-way work have good occasion to appreciate.

The top of the boiler comes so close to the clearance limit that there is very little height left for the dome and stack. The old idea of the steam dome was that it was a reservoir for dry steam on top of the boiler from which the locomotive's supply could be drawn. But the steam domes of the present day are too small to have any real effect in storing steam. This dome is little more than a foot high, and nearly half its interior is taken up by the throttle and throttle-pipe.

A rough computation shows that this locomotive when running at 60 miles per hour and cutting off at one-fourth the stroke will draw from the boiler about 1,300 cu. ft. of steam per minute. As the opening from the boiler into the dome has considerably less than 2 sq. ft. net area, this means that the velocity of the steam entering the dome through this opening will be over 700 ft. per minute, or over 11 ft. per second, and taking the net cubic contents of the dome at 3 cu. ft., a dome full of steam will be used up in one-seventh of a second.

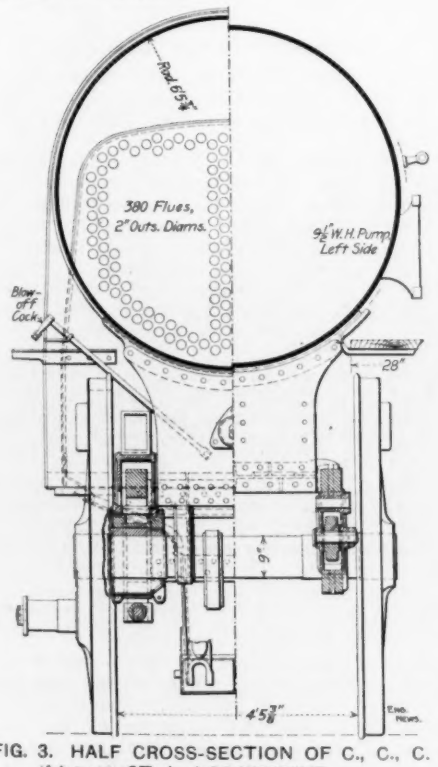

FIG. 3. HALF CROSS-SECTION OF C., C., C. & ST. L. LOCOMOTIVE.

Under these circumstances it seems to us a pertinent question whether domes on such locomotives are of any real benefit.

The firebox is of noticeable width, and has the very large area of 97 × 76 ins. Two firedoors are provided and are, of course, a necessity to enable the fireman to keep so wide a grate covered. An idea of the strenuous life which the coal-heaver on this locomotive is expected to lead may be gained from the crosswalls on the tender, indicating the way the coal will be piled up on it. Regarding this, however, it will be noted that the tender floor is set pretty high to suit the height of the boiler and also that a generous water space on the tender is another provision.

One incidental advantage of the modern type of boiler over the old is the very small area of firebox plates left unlagged and exposed to radiation. With the old type of deep firebox placed between driving wheels or between frames, there was no room to apply lagging to these plates, and the loss of heat, in severe weather particularly, was considerable.

The heavy plates used in the boiler have already been referred to. The seams are of particularly heavy proportion. The horizontal seams are quintuple and sextuple riveted and the circumferential seams are triple riveted and 1⅛-in. rivets are used. All rivet holes in the firebox are reamed after punching. The staybolts are 1 in. and 15-16-in. in diameter, and are drilled on the outside with a 3-16-in. sole 1¼ ins. deep.

We append a table of dimensions of this locomotive arranged in our standard form:

Driving wheels, diameter	6 ft. 6 ins.
Driving wheel centers	Cast-steel
Driving axles, journals	9¼ × 12 ins.
WHEELBASE: Driving	7 ft. 6 "
Total engine	28 " 5 "
Tender	18 ft.; engine and tender 55 " 3⅞"
WEIGHT IN WORKING ORDER:	
On driving wheels,	100,600 lbs.
On truck wheels	86,000 "
Engine, total	186,600 "
Tender, empty	49,320 "
Water in tank	50,000 "
Coal on tender	20,000 "
CYLINDERS: Diameter and stroke	20½ × 26 ins.
Cubic contents of both cylinders	9.93 cu. ft.
Ports, steam,	1⅞ × 25.55 ins.
Bridges, width	3 "
Valves, type	Piston; maximum travel 5⅜"
Inside lap	0 ins.; outside lap 1⅛"
Lead (full forward gear)	1-16-in
BOILER: Type	Radial stayed, Wagon top.
Diameter barrel, inside smallest ring	6 ft. 1 in.
Thickness, barrel plates	¾, 13/16, 9/16, 9/16-in.
Horizontal seams	Quintuple and sextuple.
Circumferential seams	Triple
Height from rail to center line	9 ft. 8 ins.
Working steam pressure	200 lbs.
FIREBOX:	
Length inside	8 ft. 1 in. ; width inside 6 ft. 4 ins.
Depth at front	6 " 6½ ins.; depth at back 5 " 9½"
Thickness—side, back and crown plates, each	⅜-in.
Tube plate	⅝-in.
Stay bolts, diameter	15/16 and 1 in.
Water spaces, width, front, 4 ins.; back and sides,	3½ ins.
TUBES: Material, Brooks No. 11 B.W.G.; No.	380
Pitch	2 15/16 ins.
Diameter, outside	2 ins.
Length over tube plates	16 ft. ⅛-in.
Total area of tube openings	8.29 sq. ft.
HEATING SURFACE AND GRATE AREA:	
Heating surface, tubes (interior area)	3,164.96 sq. ft.
" " firebox	175.1 "
" " total	3,340.0 "
Grate area	51.66 "
Ratio of total heating surface to grate area	64.6
Interior tube area to firebox heating surface	180.7
Sq. ft. of total heatg surf. to 1 cu. ft. cyldr volume	336.3
Firebox heatg surf. to 1 cu. ft. cyldr volume	17.6
Tube heating surf. to 1 cu. ft. cyldr volume	318.8
Grate area to 1 cu. ft. cylinder volume	5.2
MISCELLANEOUS:	
Exhaust nozzle, diameter	5½ ins.
Smokestack, height of top above rail	14 ft. 11 ins.
Capacity, tender tank	6,000 galls.
Capacity of coal space	20,000 lbs.
Total adhesive tractive power at ¼ of weight on driving wheels	25,060 lbs.
Ratio of tractive force to weight on driving wheels	0.238

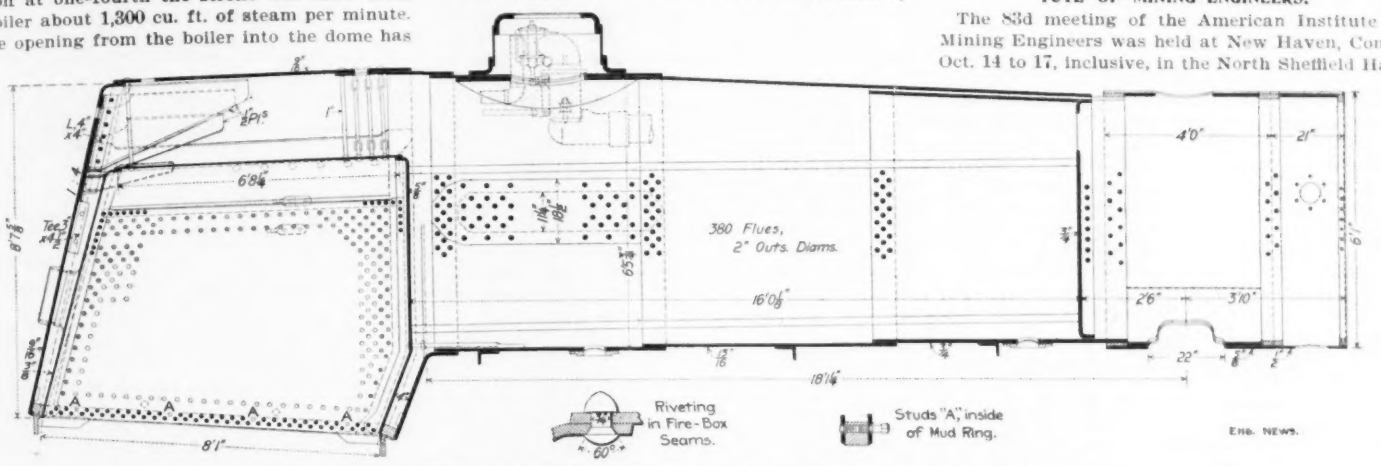

FIG. 2. LONGITUDINAL SECTION OF BOILER FOR C., C., C. & ST. L. LOCOMOTIVE.

THE NEW HAVEN MEETING OF THE AMERICAN INSTITUTE OF MINING ENGINEERS.

The 83d meeting of the American Institute of Mining Engineers was held at New Haven, Conn., Oct. 14 to 17, inclusive, in the North Sheffield Hall, Yale University. At the opening session, Tuesday evening, President Hadley, of Yale University, welcomed the Institute, and President E. E. Olcott responded. Then followed a most interesting biographical sketch of the late Clarence King by Secretary Rossiter W. Raymond, who had for many years been a co-worker and intimate friend. Dr. Raymond's splendid eulogy of King and his life's work was particularly opportune in that King was himself a Yale man. The picture of King as drawn by Dr. Raymond was both vivid and inspiriting, while the story of his early life in the far West reads like a romance. We regret the lack of space that limits us to mere mention of the merit of this biography, for nothing so stimulates men to succeed as does the sight or the story of success.

At the Wednesday morning session, Mr. Christopher G. Atwater was to have read a paper on "The Development of the By-Product Coke Ovens in the United States," but owing to the inability to show the accompanying lantern slides the paper was postponed until the evening session, and Mr. Raymond took the floor to discuss the subject of "Mine Valuation." We reserve for a subsequent issue an abstract of this paper and remarks thereon.

The afternoon was devoted to visits to some of the University buildings, and to the Yale-Vermont football game in the Athletic Field.

At the evening session, Mr. Atwater presented his paper on "By-Product Coke Ovens in the United States." The Otto Hoffman process was described and illustrated by lantern slides. The extent of the use of the process in this country is shown in the accompanying table, taken from a paper by F. Schniewind, Ph. D., read before the Engineering Congress, at Glasgow, a year ago:

By-Product Coke Ovens Erected or in Course of Construction by the United Coke & Gas Co.

	Erected.	No. of ovens.	Coke used for.	Gas used for.
Cambria Steel Co., Johnstown, Pa.	1895 & 1898	160	B*	Fuel
Cambria Steel Co., Johnstown, Pa.	1902	100	B	"
Pittsburg Gas & Coke Co., Glassport, Pa.	1896	120	B & D	"
New England Gas & Coke Co., Everett, Mass.	1898	400	D & L	Illuminating
Dominion Iron & Steel Co., Sydney, C. B.	1900	400	B	Fuel
Hamilton Otto Coke Co., Hamilton, Ont.	1900	50	F & D	Illuminating
Lackawanna Iron & Steel Co., Lebanon, Pa.	1901	232	B	Fuel
Lackawanna Iron & Steel Co., Buffalo, N. Y.	1901	564	B	Fuel
South Jersey Gas Co., Camden, N. J.	1901	100	F & D	Illuminating
Maryland Steel Co., Sparrows Point, Md.	1901	200	B	Illu'ing & Fuel
Michigan Alkali Co., Wyandotte, Mich.	1901	15	l. b.	Fuel
Sharon Coke Co., Sharon, Pa.	1902	212	B	Fuel
Total	....	2,553	..	.....

*B, blast furnace; D, domestic; L, locomotive; F, foundry; l. b., burning lime.

About 4,000,000 net tons of coal per annum are now carbonized in these ovens. The plant at Everett, Mass., produces 1,400 long tons of coke per day, of which the railroads purchase 700 tons for use in locomotives, 350 tons are sold to steam plants, and 350 tons for domestic purposes. About 6,500,000 cu. ft. of illuminating gas is sold to various gas works in Boston. It averages 19 c. p. without enriching. A ton of coking coal produces about 9,000 cu. ft. of gas, of which the earlier portion, or 4,000 cu. ft., is illuminating gas, which is sold, and the remainder, 5,000 cu. ft., is a heating gas, which is used to heat the ovens. Tar, ammonia, and other chemical by-products are also made.

In the United Kingdom 8% of the coal is coked in by-product coke ovens, and 12% is used in blast furnaces directly with the recovery of by-products. Therefore, 80% of the coal coked is with a loss of by-products. This is in accordance with the estimate of Mr. George Beilby for 1898. The American figures given by the "Mineral Resources" for 1901 show that 5% of the coal was coked in by-product ovens and 95% in beehive with the loss of by-products. In Germany the figures are only an approximation. The Otto company has built over 13,000 coke ovens to date, of which practically 40% are by-product ovens. Therefore, it may fairly be said that 60% of the coal coked was with a loss of by-products. The ovens now in operation and course of construction in the United States and Canada amounted to 3,413.* When in operation they will add about 3,000,000 net tons of coke annually to the present by-product coke production, which will then constitute 13% of the total coke production.

The by-product coke ovens first built in this country were copied directly from those in successful operation abroad, but conditions under which they operated in this country have produced a development on different lines. The original type of the Otto-Hoffmann, or vertical flue oven, as built for the Cambria Steel Co., at Johnstown, has been reproduced with but little deviation at the succeeding plants in Glassport, Sydney, Hamilton, Camden and Lebanon. The characteristics of this design were the vertical flues and the use of regenerators. The improvements consisted of automatic methods in coal and coke handling; the development of electrically-operated labor-saving devices and improvements in construction details.

The principle of division of gases to obtain a high power illuminating gas from the ovens had also been developed by Dr. F. Schniewind, notably at the Everett and Hamilton works.

The use of coal compression as a method of dealing with some varieties of coal in order to obtain an improved coke has been tried at Sydney with success.

The success of the under-fired principle, as introduced by the builders of the Otto-Hoffmann ovens in Germany, established the value of this method of firing, and led to the modification of the type of oven to embody this principle, together with the use of regenerators and the consequent economy in gas consumption. The Schniewind type of oven was thus developed, of which the characteristics were the use of the under-fired principle and the entire separation of the regenerative chamber from the oven-supporting structure, embodying the advantages of both systems. Ovens of this form are being installed at Wyandotte, Sparrow's Point, Sharon and Johnstown.

Reference was also made to experiments on the progress of coking in the oven charge taken from data obtained in Germany and in Sydney, showing that the center of the oven charge remains at a low temperature until about half the coking time has elapsed, the gas escaping up the walls of the oven and through the coked portion.

The rest of the session was taken up by an illustrated lecture by the veteran mining engineer, Prof. Wm. P. Blake, of Tucson, Arizona, on the "Blake Crusher and Its Application in Various Industries." Prof. Blake is a relative of Eli Whitney Blake, who invented the crusher, or as he preferred to call it, the stone-breaker, about 50 years ago. The lecturer showed drawings of the early forms, which were practically as good as the later ones, and of several infringing devices, some of which were highly ingenious but practically inefficient. The great saving of labor which was accomplished by the Blake crusher, making possible the treatment of low grade ores and the cheap construction of macadam roads, of concrete, etc., warrant its being classed as one of the great inventions of the 19th century.

*It will be noted that this figure represents a considerably later date than the figures in the table.

At the session on Thursday morning, Prof. H. E. Gregory, of Yale University, gave an illustrated lecture on the "Geology of the New Haven Region." It was highly interesting, and showed in the clearest manner the remarkable geological features of the region, including the intrusion of vast trap dykes into the sandstone, as shown by the exposures on East and West Rocks, near the city, and the effect of glacial action in the valley.

The lecture was followed by an exhibition by Dr. Leonard Waldo, of New York, of a new optical pyrometer. The new instrument appears to mark the most important advance in pyrometry that has been made since the invention of Le Chatelier's electrical pyrometer. The optical pyrometer is used for determining the heat in a furnace by observing the color of the interior walls. It depends on the principle that the color of a heated brick is an indication of its temperature. A telescope supported on a tripod several feet away from the furnace is pointed towards the aperture in the furnace, and the color of the interior thus observed. Inside of the telescope there is a very small carbon-filament electric lamp, supplied with current by two dry battery cells through a graduated rheostat.

When there is not enough current passing into the lamp to make the filament visibly red, it appears to the observer at the telescope as a black line on the hot fire brick of the furnace. As the current is gradually increased the black line changes to dull red, then bright red, and then to the color of the heated brick, when it becomes entirely invisible. Still more current makes it visible again as a line of lighter color than the brick. When the current is so adjusted that the filament is invisible, on account of its having the same color as the brick, the strength of the current is read on a millivolt meter, and from this reading, by reference to a table of comparison, the temperature is read. The limit of the apparatus is the temperature at which the carbon filament will be destroyed, which is above 2,000° C. (3,600° F.), or more than 200° C. above the temperature of melting platinum. For standardizing the instrument a small gas furnace, in which the temperature is determined by a Le Chatelier pyrometer, is used. After a description of the apparatus, Dr. Waldo invited his audience to take turns in examining it in operation. He had a small gas furnace in which was placed the platinum-rhodium couple of the Le Chatelier pyrometer used for standardizing, and a short distance away the tripod with its telescope containing the electric light. The remainder of the apparatus was only two dry-battery cells, a rheostat, and a millivolt meter. The furnace was heated to a high temperature by a forced blast gas burner, and the gradual change of color of the carbon filament until it became invisible, as the current in the lamp was increased, was clearly observed.

In the afternoon an excursion was made by trolley car to West Rock, which was climbed on foot. It stands 300 ft. above the plain. The exposures of the trap rock, and its lines of cleavage at right angles to the plane of the sandstone beneath were pointed out and discussed by the geologists. On top of the rock there are some enormous boulders, carried there by the glaciers. They form a cave which is of historical interest, as it was for some weeks the hiding place of two of the "regicides," the judges who condemned Charles I. to the scaffold, and on whose heads a price was set when Charles II. ascended the throne.

In the evening a reception given by the local committee was held at the Yale Art School.

The printed programme of the meeting announced a session for Friday morning, and visits to the University buildings and the Peabody Museum for Friday afternoon; but at the close of the session on Thursday morning it was announced that there would be no further session, and that all of Friday would be available for visits to the buildings. Upon this announcement many of the members left for home on Thursday evening, including the present writer.

It cannot be said of this meeting, as used to be said of the meetings of the Institute many years ago, "this is the best meeting yet." In fact, it was one of the poorest. Only 80 members and guests had registered by Wednesday evening, including 18 ladies and 21 members of the local committee. The programme of papers to be read was not obtainable until the members arrived at the meeting; the papers were not printed in advance, and there were no discussions. The list of papers "to be presented at the meeting," as the programme says, and which will finally appear in the printed volume of Transactions, is as follows:

1. Biographical Notice of Clarence King, by R. W. Raymond, S. F. Emmons, James T. Gardiner, and James D. Hague.
2. The Development of By-Product Coke Ovens in the United States, by Christopher G. Atwater, New York city.
3. The Blake Crusher and Its Applications in Various Industries, by W. P. Blake, Tucson, Arizona.
4. The Lodes of Cripple Creek, Colorado, by T. A. Rickard, Denver, Colo.
5. The Veins of Boulder and Kalgoorlie, by T. A. Rickard, Denver, Colo.
6. Preliminary Notes on Vein Enrichment by Ascending Hot Waters, by Walter Harvey Weed, Washington, D. C.
7. Ore Deposits Near Igneous Contacts, by Walter Harvey Weed, Washington, D. C.
8. What is a Mining District? by R. W. Raymond, New York city.
9. The Valuation of Mines of Definite Average Income, by H. D. Hoskold, Buenos Ayres, S. A.
10. The Geology of Southwest Texas, by E. T. Dumble, Houston, Texas.
11. Notes on the Treatment of Zinc-Precipitate Obtained in Cyaniding New Zealand Ores, by Hamilton Wingate, Bristol, England.
12. The Direct Cyaniding of Wet-Crushed Ores in New Zealand, by Hamilton Wingate, Bristol, England.
13. The Copper Deposits of Sierra Oscura, New Mexico, by H. W. Turner, San Francisco, California.
14. Amarillium, by Wm. M. Courtis, Detroit, Mich.
15. The Manganese Industry of the Department of Panama, Republic of Colombia, by E. G. Williams, Colon, S. A.
16. An "All-Fire" Method for the Assay of Gold and Silver in Blister-Copper, by Walter G. Perkins, Grand Forks, B. C.
17. Coking in Bee-Hive Ovens with Reference to Yield, by Charles Catlett, Staunton, Va.
18. Elimination of Arsenic, Antimony and Bismuth from Copper, by Allan Gibb, Mount Perry, Australia.
19. The Chemistry of Ore-Deposition, by Walter P. Jenney, Salt Lake City, Utah.
20. The Measurement of High Furnace Temperatures, by Leonard Waldo, New York city.

THE MEANING OF "ORE IN SIGHT."

In the reports of mining engineers the expression "ore in sight" is a common one, but one that appears to have had no definite, uniform significance.

Mr. C. McDermid, Secy. Institution of Mining and Metallurgy, Salisbury House, London, E. C., has favored us with a copy of the findings and recommendations of the Council of the Institution on this subject, which we quote in full:

1. That members of the institution should not make use of the term "ore in sight," in their reports, without indicating, in the most explicit manner, the data upon which the estimate is based; and that it is most desirable that estimates should be illustrated by drawings.
2. That as the term "ore in sight" is frequently used to indicate two separate factors in an estimate, namely: (a) Ore blocked out, that is, ore exposed on at least three sides within reasonable distance of each other. (b) Ore which may be reasonably assumed to exist though not actually "blocked out." These two factors should in all cases be kept distinct, as (a) is governed by fixed rules, whilst (b) is dependent upon individual judgment and local experience.
3. That in making use of the term "ore in sight" an engineer should demonstrate that the ore so denominated is capable of being profitably extracted under the working conditions obtaining in the district.
4. That the members of the institution be urged to protect the best interests of the profession by using their influence in every way possible to prevent and discourage the use of the term "ore in sight" except as defined above; and the council also strongly advise that no ambiguity or mystery in this connection should be tolerated, as they (the council) consider that such ambiguity is an indication of dishonesty or incompetency.

The third recommendation of the Council appears to us superfluous, for the mining expert, in his report upon a property, always gives a valuation or assumed valuation of the "ore in sight," in conjunction with an estimate of the cost of mining and treating the ore, from which data any business man is capable of judging whether the "ore in sight" is worth mining or not. It seems to us much more important that the mining engineer make clear that the estimate of value is based upon a sufficiently large number of assays to warrant using such an estimate as the average value of the "ore in sight."

Furthermore, in restricting the use of the term

"ore in sight" to ore that it will pay to mine, the Council appears to have lost sight of the fact that low grade ore is frequently mined along with the higher grade paying ore, and that this low grade ore is often stored in dumps with the express intention of treating it, years later, perhaps, when freight, milling or smelting rates shall have been lessened. Such low grade ore is, nevertheless, "ore in sight," and a probability amounting

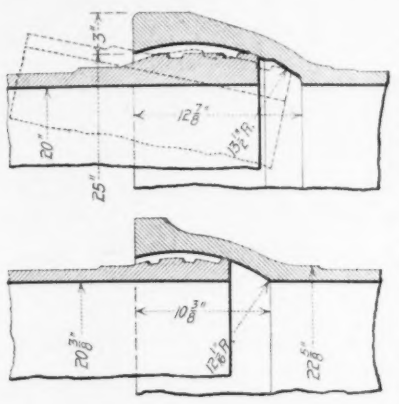

Fig. 2. The New Form.

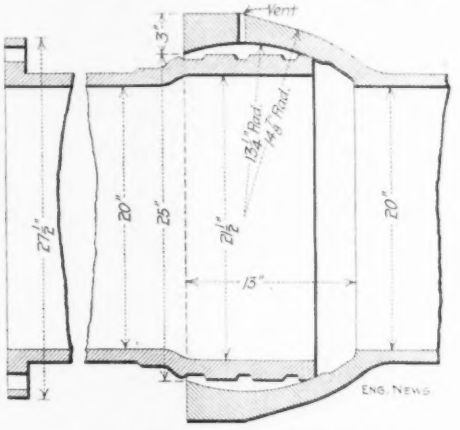

Fig. 1. Two Old Forms.

NEW AND OLD FORMS OF BALL JOINTS FOR SUBMERGED 20-IN. CAST-IRON WATER MAINS AT ATLANTIC CITY, N. J.

to a certainty may exist that it can be profitably treated when the growth of the mining district shall have brought competitive railways, mills or smelters to the district.

FORMS OF BALL JOINTS FOR SUBMERGED WATER MAINS AT ATLANTIC CITY, N. J.

Some years ago there was laid at Atlantic City, N. J., some submerged 20-in. cast-iron water pipe, with ball joints of the forms shown by Fig. 1. These joints proved unsatisfactory, and in recent submerged work the form shown by Fig. 2 has been used. It will be noticed that the new joint has a more concave bell than the old one, thus affording a firmer grip for the lead.

Two lines of 20-in. submerged pipe, with a joint similar to the new one shown herewith, were completed during the past summer, and promise to be satisfactory. The new joint herewith is being used on a contract awarded Sept. 2, 1902, for taking up old pipe, saddles and piles in Beach Thoroughfare, and laying new pipe, with piles and saddles. The contract price is $2.90 per lin. ft. for taking up the old line, and $7.90 per lin. ft. for constructing the new. The contractors are Van Sant & Boehm, of Atlantic City, who offered to do the work under a five-year guarantee for $9.25, or $1.35 per lin. ft. extra.

We are indebted for the above information, including illustrations, to Mr. Kenneth Allen, M. Am. Soc. C. E., Engineer and Superintendent of the Atlantic City Water-Works. Mr. Allen states that in taking up old submerged pipe "it was found that a number of joints simply pulled apart by the weight of the pipe." He also says that trouble with the old main was "probably made worse by poor alinement of the pipe, some lengths of which were not in the saddles." The joints now being used were designed by Mr. Allen's predecessor, Mr. W. C. Hawley, Assoc. M. Am. Soc. C. E., now located at Wilkinsburg, Pa.

TESTS TO DETERMINE THE PROTECTION AFFORDED TO STEEL BY PORTLAND CEMENT CONCRETE.*

By Prof. Charles L. Norton.†

The constantly increasing use of steel as a structural member in modern buildings has led to many questions as to the permanency of the steel as sometimes used for this purpose. The examination of buildings ten to fifteen years old, when, during alterations, the steel framework has been exposed to view, reveals all stages and conditions of disintegration of the steel. So great has been the corrosion, even in this short time, in some cases, that a note of alarm has been sounded by some engineers most familiar with the subject. The use of steel beams and posts is of so recent date that no very exact deduction can be drawn as to the exact time required for a very serious or destructive loss of steel through corrosion; but surely, when a steel plate ½-in. in thickness loses more than ⅛-in. in five years, there arises a question as to the ability of the structure to last more than 25 years.

Some of the factors in the matter of corrosion of the steel we know; others we do not know, and cannot until after a lapse of some years.

There can be no question that moisture and carbon dioxide are the active agents in causing much of the rusting of the steel. To what an extent the two are relatively responsible and in what measure they need renewal, to keep up the process, is uncertatin. It has been held that the formation of a coat of rust upon the surface of steel was the beginning of a progressive action whereby the rust, or iron oxide, acted as a continuous carrier of oxygen to the steel beneath. This process seems to require only moisture and atmospheric air containing carbon dioxide to start it, but as to the depth of penetration of the process I know of no assignable maximum in any given time. It is extremely probable that in a comparatively dry place the process is exceedingly slow.

There can, of course, be no question as to the ease of access to the steel, in many cases, of both moisture and carbon dioxide. When steel is bedded in the wall of the building, as is almost always the case, the changes in temperature from time to time, as well as the more or less constant difference in temperature between the two faces of the wall, tends to cause a condensation of moisture in the wall at different points. Further, the necessary carbon dioxide is most plentiful in the large cities, where the steel frame is most common.

When the walls of the building are of brick or stone, moisture and carbon dioxide may usually enter at the joints and, to a greater or less extent, through the body of the stone or brick. Few stones are, however, porous to such an extent as to allow an appreciable penetration. Terra cotta tile is of itself porous, and the existence of air passages tends to increase the condensation of moisture and absorption by the terra cotta and possible contact with the steel. Concrete made of Portland cement with sand and either cinders or stone, would seem to offer more protection to the steel than any of the materials just mentioned; yet, we hear from time to time of the loss, by corrosion, of steel bedded in concrete.

A study of the action of Portland cement concrete on steel was begun under my direction by Mr. P. C. Pearson in December, 1901, and it is my purpose to report upon this and some later observations as a preliminary to more work of the same nature now under way.

It has been held by several engineers that the mere alkaline nature of Portland cement was a sufficient guarantee of its protecting steel from rusting. There is, of course, good chemical reasoning for this, the familiar use of strong alkaline solution in boilers to prevent the formation of scale being based upon the same principle. This would seem to settle the matter once and for all, were it not a fact that steel bedded in concrete has corroded very rapidly, while other steel in a different concrete of the same kind of cement stands without change for ten years or more.

The investigation was started not to find out that steel could not be protected by concrete or that it could be, for on that point we were sure, to start with, but we have tried to find a reason for occasional failure and to suggest a remedy..

An examination of several cases where expanded metal had been embedded in concrete showed plainly that wherever the steel was exposed through cracking rusting began, even though the cracks were very fine. It would seem that the alkaline nature of the cement would be sufficient to prevent corrosive action occurring within a few hundredths of an inch on the moist surface of the steel, but such is not always the case.

To study the matter systematically, two brands of American Portland cement (Alpha and Lehigh) were selected; two kinds of cinders, one from the Sugar Refinery, the other from the Boston & Albany R. R. locomotives; a sharp, clean beach sand; and a hard, clean broken stone, the larger part being fragments of flint and trap rock. Concretes were made up in bricks about 3 × 3 × 8 ins., with the steel specimens near the center.

The following mixtures were tried at first; others will be reported on later: Neat cement; cement, one part, to three of sand; one part of cement and five parts of broken stone; and one part of cement to seven parts of cinders. All briquettes were made in duplicate with both cements. There were later made up briquettes of one part of cement to two parts of sand and five parts of cinders, and of one part cement to two of sand and five of crushed stone. The table below gives a summary of the mixtures:

Alpha cement.	Lehigh cement.	Sand.	Stone.	Cinders.
1	0	0	0	0
0	1	0	0	0
1	0	3	0	0
0	1	3	0	0
1	0	0	5	0
0	1	0	5	0
1	0	2	5	0
0	1	2	5	0
1	0	0	0	7
0	1	0	0	7
1	0	2	0	7
0	1	2	0	7

It was hoped to vary the density, the porosity and the nature of the contact with the steel, as well as the chemical composition of the concretes. The cements were tested chemically and physically and found good. The cinders, when washed down with a hose stream and dried, tested distinctly alkaline, and analysis revealed very small amounts of sulphur. The stone and sand were thoroughly washed and clean. The ingredients were mixed dry in every case and, when wet, thoroughly mixed and tamped until wet on top.

The cleaning of the steel was the most troublesome problem met with. It was necessary to scour the pieces, then pickle in hot dilute sulphuric acid, and finally dip into hot milk of lime. When cold, the lime was removed with a wire brush. This left the steel clean and bright, ready to be put into the test bricks.

The speciments used were a mild steel rod 6 ins. long and ¼-in. in diameter, a piece of soft sheet steel 6 × 1 × 1-32 ins., and a strip of expanded metal 6 × 1 ins., all three pieces being put in each brick. Since time would not permit of our exposing these speciments to natural conditions, we enclosed them in several large tin boxes,

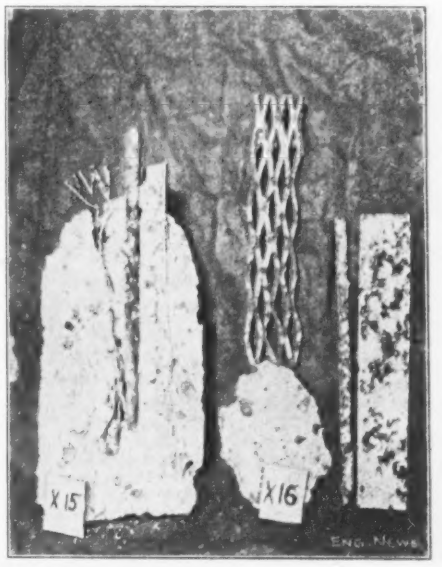

Fig. 1. Showing Steel Rusted by Contact with Cinders.

sealed tightly, and subjected one-quarter of them to an atmosphere of steam, air and carbon dioxide, and second quarter to air and steam, a third to air and carbon dioxide, and a fourth stood upon the table of the room, with no special care as to their temperature or dryness. Of the entire number, about one-half were set in water for one day, the rest for seven days, before sealing up.

At the end of three weeks the briquettes were carefully cut open and the steel examined and compared with specimens which had lain unprotected in each of the tin boxes.

Those specimens which were mixed of neat cement can be dismissed without discussion, for the protection was

*Abstracted from Report III. of the Insurance Engineering Experiment Station, Boston, Mass.

†Engineer in Charge, 31 Milk St., Boston.

perfect. The steel was as bright as when put in. The unprotected pieces were found to consist of rather more rust than steel. The steel was wrapped about pieces of uralite, to serve as a means of identifying it by number, the stamped numbers being nearly obliterated by the rust. Of the remaining specimens, hardly one had escaped serious corrosion. The location of the rust spot was invariably coincident with either a void in the concrete or a badly-rusted cinder.

In the more porous mixtures, the steel was spotted with alternate bright and badly rusted areas, each clearly defined. In both the solid and the porous cinder concrete, many rust spots were found, *except where the concrete had been mixed very wet, in which case the watery cement had coated nearly the whole of the steel, like a paint, and protected it.*

Some briquettes made later of finely ground cinders and cement, in varying proportions, when exposed to moisture and carbonic acid, showed how effectually the presence of cement prevented rusting, even in a highly porous mass—one cement to ten of cinder, provided there were no cracks or crevices or distinct voids. From the examination of these several hundred briquettes we were able to draw several conclusions:

(1) Neat Portland cement, even in thin layers, is an effective preventive of rusting.

(2) Concretes, to be effective in preventing rust, must be dense and without voids or cracks. They should be mixed quite wet where applied to the metal.

(3) The corrosion found in cinder concrete is mainly due ot the iron oxide, or rust, in the cinders, and *not* to the sulphur.

(4) Cinder concrete, if free from voids and well rammed when wet, is about as effective as stone concrete in protecting steel.

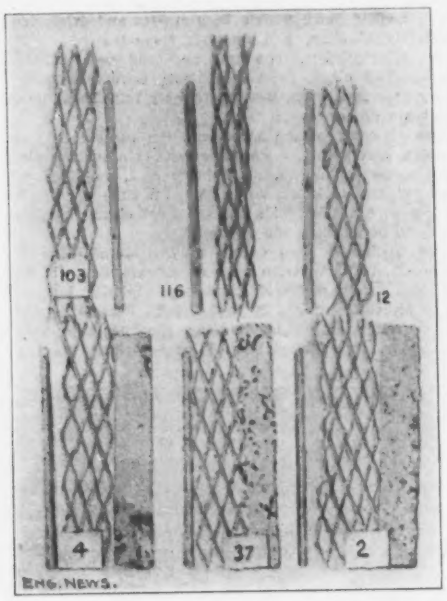

Fig. 2. Showing Steel Rusted at Voids in Concrete.

(5) It is of the utmost importance that the steel be clean when bedded in concrete. Scraping, pickling, a sand blast and lime should be used, if necessary, to have the metal clean when built into a wall.

In the matter of paints for steel, there is a wide difference of opinion. I cannot believe that any of the paints of which I have any knowledge can compare with a wash or painting with cement. Moreover, if paint does disintegrate, it leaves a thin void next the steel, the worst possible condition. I examined some steel Z bars not long ago, which were exposed in process of altering a building five years old. It was clear that the steel was rusty when painted and that moisture had penetrated through the paint, caused the rust to increase and split off the paint. This was a case where the steel was built into the brick work with no attempt at any protection other than the paint.

It is, in my opinion, perfectly clear that the coating of all steel work with cement before applying the concrete or tile or brick, as the case may be, is an absolute essential, if the formation of rust and consequent weakening of the steel is to be prevented. The thickness of the cement layer need not be great, but it should be a continuous coating, without cracks. The steel should be perfectly clean; but if, as is often the case, the choice is between paint and rust which accumulates during construction, the paint is to be preferred.

Mr. Pearson suggested the use of Portland cement in protecting the under side of railway bridges against rust and the blasting action of the gases and cinders from locomotives. I think this merits a careful trial.

It is intended to take up the study of paints as a protection for steel, and to study, as well, the action of corrosion with long lapses of time in the walls and buildings of the Experiment Station now about to be erected.

COLUMNS FOR BUILDINGS.

By John Stephen Sewell, Captain, Corps of Engineers,* U. S. A.

This is not, as might be inferred from its title, an attempt to treat the whole of a very broad subject; on the contrary, it is only a series of suggestions relating to certain questions raised directly or indirectly by the editorial in Engineering News of August 28, 1902, on the Box Column. Had the title been sufficiently specific to define the limits of what the writer has to say, it might have been longer than the paper itself.

The box column presents a section which is almost ideal in the distribution of metal about the center of gravity. That it lends itself well to the details commonly used in steel frame buildings is amply demonstrated by the numerous excellently designed buildings in existence in which it has been selected. If it were considered necessary to fully splice the columns in a building, however, it is probable that the box column would speedily go out of use, at least for heavy sections; for such splices would be very awkward to design and execute; but as long as it is necessary only to hold the ends of the column in line, splicing box columns is very simple. Many managers of structural shops have told the writer that they preferred to drive the four rows of rivets in a box column, rather than the two rows in a Z-bar column, because of the greater facility of getting at them. Of course, in heavy sections, where cover plates would be used on the Z-bars, this difference becomes much more marked; it may be concluded, therefore, that the box column is at least as economical as any in fabrication.

The only one of the objections which may have been urged in the past against the use of box columns, and which seems plausible at the present time, is the danger of internal corrosion. Possibly this may be a real danger in some cases, but in buildings, as a rule, it is probably not imminently serious, and it can be easily eliminated altogether at small cost by filling the columns with concrete.

It has not been the writer's privilege to examine many hollow columns, except some cast-iron ones, after their removal from old works; but he has seen a good deal of rolled iron and steel in various shapes, taken from old buildings where the conditions certainly seemed favorable for corrosion. All that he has seen tends to confirm the statements made by Mr. Szlapka, in Engineering News of September 11, 1902.

However, it would seem that we have sufficient data to enable us to prevent the corrosion, even in the exceptional cases where the conditions would otherwise insure its rapid progress. In Bloxam's "Chemistry," edition of 1895, pp. 376 and 377, it is stated that the ordinary rusting or corrosion of iron is accomplished only in the presence of moisture, air, and CO_2. If any of these substances is absent, the change cannot take place. The reactions are given as follows:

$$Fe + H_2O + CO_2 = FeCO_3 + H_2. \quad (a)$$

The $FeCO_3$ is dissolved by the carbonic acid present, and the solution absorbs oxygen from the atmosphere, in accordance with,

$$2FeCO_3 + O = Fe_2O_3 + 2CO_2; \quad (b)$$

the Fe_2O_3 combines with the water and is deposited as $2Fe_2O_3 . 3H_2O$, or ordinary iron dust.

Bloxam further states that iron in its ordinary state is not affected by perfectly dry air, and that it will not rust in water containing a free alkali or alkaline earth or an alkaline carbonate. If this is true, it is probably because the affinity of these alkaline substances for any acid is greater than that of iron, so that they would neutralize the acid before it had time to attack the iron.

It is evident from a consideration of the reactions given above that if they are correct it is only necessary to have a limited amount of CO_2, with a continuous supply of moisture and air, to effect the ultimate destruction of a large amount of iron, because the CO_2 never becomes fixed, but remains ever active, and passes on from molecule to molecule, as long as there is any free Fe for it to attack. It is probable that any other of the common acids would act in practically the same way, and experience seems to indicate that this is true.

Practical men, as a result of observations on painted iron or steel structures, often assert that iron rust once started is like a cancer in that it eats its way ever inwards, and is not stopped by an outer covering of paint. The reason probably is that in spite of the paint, enough moisture and oxygen reach the diseased spot to maintain the activity of the CO_2 or other acid. Now if alkaline substances really have the protective power indicated by Bloxam, it is evident at once that a substance so strongly alkaline as Portland cement, even after hydration, should protect steel against corrosion under ordinary conditions, and there is no lack of examples to show that it does do so.

To be effective, the contact between the steel and cement should be perfect, or else the conditions should be such that any moisture reaching the steel must be filtered through the alkaline mass and thus robbed of its acid before reaching the steel. Even if corrosion has set in before the cement is applied, the CO_2 concerned in it would be speedily fixed by the alkaline moisture accompanying the cement and thus rendered harmless. The easiest way to apply cement to columns is in the form of concrete filling within the columns proper and between them and the fireproof covering. It is an additional safeguard to increase the alkaline earths present by the addition of a little slaked lime, say by mixing the concrete with whitewash instead of plain water. As this does not detract from the strength and durability of the concrete and costs only a nominal amount, it is well always to do it where protection of steel against corrosion is one of the objects.

The writer has great confidence in the protective power of real Portland cement concrete under most conditions, but he would hesitate to use slag cement for this purpose because of the sulphides apt to be present; and he believes that grillages immersed below the level of the ground water will not be permanently protected, chiefly because of the prevalent careless method of depositing the concrete. This leaves large voids through which the acid-charged water can reach the steel without being filtered. In case the water contains much acid, and flows, as most ground water does, all the alkali in even the best concrete might ultimately be neutralized, and then the acid would be free to attack the steel. For these reasons, if any of our steel frame buildings should fail through corrosion, the writer expects to see the failure begin with the grillages, in most cases; and to see it manifested by the columns punching through, long before they themselves are seriously weakened.

If protection against corrosion were the only advantage of concrete filling in columns, it might be doubted whether it is worth while in the cheaper grades of commercial work, though always advisable in permanent public works and the better grade of private works. But there are additional advantages which, it is thought, justify it in every case. A column filled with good concrete must be—in fact is—very much stronger and stiffer than one without such filling, and it would buckle much less easily in a fire than an empty column of the same size. The writer would like to be in a position to give positive and definite data on these points; unfortunately, he is not, but he feels quite sure that his statements are, in the main, correct. Some rather expensive experiments would be needed to demonstrate their truth, and no one has as yet made them. However, there are sufficient data to indicate that concrete has great stiffening power. Among the works executed by one of the companies now engaged in exploiting reinforced concrete are many cases of concrete reinforced against compression by bars of moderate size imbedded in the mass. In some cases these bars are so close together that

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they almost touch throughout the entire section of the composite member. They are figured in simple compression at about 16,000 lbs. per sq. in., and have nothing but the concrete to prevent them from buckling. They stand under strains that the bundle of bars alone could not stand for a moment without buckling, and yet the concrete around them seems absurdly small in amount. Imagine a column made of 15-in. channels and lattice bars, filled and surrounded with concrete so as to be, say, 24 ins. square. Surely we would be well within the limits of safety if we figured the steel in simple compression, and the column would be secure against corrosion and fire to a degree rarely seen in current work.

Again, we know that the skin friction between concrete and imbedded steel amounts to as much, on the average, as 500 lbs. per sq. in. Suppose we can assume 30 lbs. as a safe working load. In the case of the 15-in. channel column, leaving out of consideration the effect of rivet heads and lattice bars, skin friction alone might transfer from the steel to the concrete over 14 tons per lineal foot. If the column and its concrete filling and covering are designed with that end in view, the concrete will certainly transmit a large portion of the load to the foundations. The writer has seen reinforced concrete columns of ordinary proportions designed for a working load of 70 tons per square foot, standing up without sign of distress under more than their full load, in a building not at all free from vibration. Certainly all these facts indicate enormous stiffening power in the concrete, although they do not yield definite figures as to its value.

Suppose our concreted latticed column is made up of two 15-in. channels, 55 lbs. per foot, and 12 ft. long. It is probable that the average conservative designer would consider this column good for about 224 tons. If figured in simple compression, it would be good for at least 15% more, or, what is practically the same thing, its section could be reduced about 15% for the same load. This would save, say, 16 lbs. of steel per lineal foot. At 3½ cts. per lb. this would amount to 56 cts. per lineal foot. The ordinary column covering would probably come to not less than 80 cts. per lineal foot. The sum of these items is $1.36 per lineal foot, which would be available for the concrete covering and filling. This ought not to cost more than $1.00 to $1.10 per lineal foot. The completed column would thus cost less than it does in current practice; it would be probably much stronger, and certainly more thoroughly fireproof and rustproof.

There is another way of looking at the division of load between the steel and the concrete. In the case assumed, it would be fair to say that the concrete covering and filling are equivalent to a solid concrete column having a sectional area of 3 sq. ft. If we assume that owing to skin friction and other causes, the column and its concrete can act as a unit, the load will be divided between them in proportion to their moduli of elasticity. Assuming this to be as 1 to 15, the concrete at the base of a line of columns would carry about 1,000 lbs. per sq. in., or 432,000 lbs. This is at the rate of about 70 tons per sq. ft.; it seems a little high, but there are many cases where concrete is doing just such duty with no reinforcement execpt a few steel bars to prevent buckling. While the writer would hardly advocate reducing the figured loads on a steel column before designing it, because of the probable transfer of a part of it to the concrete filling and covering, the foregoing considerations seem to him to justify figuring the comparatively short columns used in buildings in simple compression under their full loads, if covered and filled as indicated. This would eliminate all question of the superiority of one section over another, so far as load-carrying capacity is concerned; it would also dispense altogether with column formulas in building work, so long as steel columns are used.

At the new building for the Government Printing Office, in Washington, D. C., of which the writer has charge as designing and constructing engineer, the columns were designed to carry their loads alone, although with somewhat higher unit strains than were assumed for girders and beams. As an afterthought, with a view of eliminating completely the danger of rust, they were filled with concrete. They were composed of channels placed back to back, with cover plates or lattice bars, according to the load. Where not built into the walls they were fireproofed with 4 ins. of brickwork and all the space between this covering and the columns was filled either with concrete or with mortar and spalls, which amounts to the same thing.

In the majority of cases this 4-in. brick covering served as a mold for the concrete. In a few cases where it was desired to fill a latticed column before the brickwork was built around it, a rough board was propped up against the side containing the lattice bars. As soon as the concrete filling had set, the board was removed. The space between the brick covering and the column was in all cases filled by the masons with mortar and spalls, as they were laying the bricks.

The aggregate used was broken brick or broken stone, broken to pass a 1-in. ring, with nothing screened out but the dust. The proportions used were about 1, 4, and 9, and whitewash was used for mixing instead of water. The concrete was put in as occasion offered, after the columns were plumbed and connected and before other work had advanced to points where it would make the concreting impossible. The concrete had to be put in a shovelful at a time, often through the lattice bars. As it usually had from 6 to 7 ft. to 40 ft. to fall, this alone was relied upon for ramming. It was made as wet as seemed safe, to secure a dense mass; but this had to be watched carefully to avoid bursting the covering on latticed columns by hydrostatic pressure. In one case, this happened, or else the moisture in the filling froze and the result was the same. The covering cracked, but was not removed until the filling was nearly a year old. It was then found to be exceedingly hard and dense, and was apparently accomplishing the object for which it was put in, so far as such a short test could indicate.

The cost of the labor involved in this work has not been finally and accurately worked out, but it is certainly less than $1.25 per cu. yd. of concrete placed, which seems reasonable enough, considering the conditions under which it had to be done. The main difficulty is to get the filling done at the proper time without delaying other work. It would be a troublesome innovation to introduce, on this account. This difficulty could be partially avoided, however, if it were considered in the design of the steelwork from the beginning.

As an example of the stiffening effect of concrete, the following experience with the columns supporting the traveling crane in the engine room of the New Building for the Government Printing Office is submitted. The track and columns were designed originally with a view to a total load of 25 tons, including the crane. Afterwards it was considered necessary to install a 25-ton crane, calling for a total load of 53 tons. Fortunately, such concentrations of the original load of 25 tons had been assumed that the track seemed good for the greater load under the actual distribution of the crane, except that the columns seemed a little scant. They are made of 6-in. extra heavy pipe, and are 21 ft. long. It was decided to fill them with concrete, and let them go. The concrete was mixed about 1, 3, 6, and rammed in place while the column was lying on its side. The crane has since been loaded with over 35 tons, making a total load of 63 tons, with the top carriage in its worst position for the columns. While it was not expected that the columns would have failed under this, it was feared that they might vibrate severely, especially when the load was moving or increasing. On the contrary, they remained remarkably rigid; hardly a tremor could be felt, and while we had no unfilled pipe columns to compare with them under the same conditions, it seems evident to all who are familiar with the conditions that the concrete filling must have increased the stiffness at least 25%. Certainly they look very slender for the loads they have to carry.

VIEW IN BASEMENT OF NEW GOVERNMENT PRINTING OFFICE BUILDING, WASHINGTON, D. C., SHOWING LATTICED COLUMNS WITH BRICK COVERING AND CONCRETE FILLING PARTLY COMPLETED.

From columns figured in simple compression and filled with concrete to prevent buckling, it is but a step to a concrete column, reinforced against buckling or even plain compression by imbedded steel bars. This seems to be the logical direction of future development, even in high buildings. It will bring the column formula once more into prominence and will no doubt give rise to many animated discussions. But the writer, for one, is convinced that it is coming, accompanied, of course, with reinforced concrete girders, beams, and floor systems, and that it will result in better and more economical designs than those current at the present time.

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ADVERTISING RATES: 20 cents a line. "Want" and "For Sale" notices, special rates, see page XXIV-XXIX. New copy for standing advertisements should be received one week in advance of publication; new advertisements, Monday morning. The pages containing "Want," "For Sale" and "Proposal" advertisements are held open until Wednesday noon.

The interesting letter on the need of standards for turbine measurement which we print in this issue deserves the attention of engineers interested in any way in hydraulic power development. We know of no department of engineering work of the importance of turbine manufacture in which reliable and valuable technical literature is so scanty; and consequently in which the engineer has less to guide him in intelligent and successful design.

It is not difficult to find a reason for this. The old-time turbine wheels were developed by rule-of-thumb mechanics to meet the conditions which attended the use of water power in the first three-fourths of the last century. During that time nine-tenths of all the water wheels in use, probably, were installed to drive country saw mills, grist mills and the like. The purchasers and users were not engineers, nor did they employ engineers. Economy and efficiency were of importance only for advertising purposes. The application of engineering science to problems of manufacturing and industry, which was so notable a feature of the period from 1865 to 1885, left the turbine business almost untouched, for the reason that the rapid development of the steam engine threatened to drive water power out of use in all large industrial undertakings. The cost of steam power became so small in most lines of manufacturing that it was cheaper to locate a plant where other considerations dictated than to place it beside a waterfall. The business of the turbine makers was confined more than ever to the supply of small grist mills and saw mills in the country, and there was no inducement for the employment of engineering talent.

In the late '80's this situation was revolutionized by the introduction of electric power transmission. Water-power developments on a large scale at once began, and have ever since continued, and turbine builders have been too busy in meeting the demand for their product and enlarging their works to keep pace with the demand to give adequate attention to problems of design and standardization.

We do not mean, of course, that manufacturers have been wholly without progress, or that engineers have entirely neglected this branch of professional work. There have been instances of good work, and certain manufacturers and certain engineers are entitled to credit therefor. The point to which we wish to direct attention is that these isolated peaks of knowledge—if we may so term them—are of little effect in raising the general level. The information which is kept secret as part of the stock-in-trade of an engineer or a manufacturer is of very little service to the world compared with that which is published and made available to all. Interchange of information is of mutual benefit, and it is because there has been a dearth of it in the turbine industry that the lack of progress set forth by our correspondent in another column has come about.

The business of this journal is the manufacture of technical literature, and in this field of hydraulic engineering where good work particularly needs to be done, we are pleased to announce that we shall shortly present a series of papers prepared for us by an engineer of large experience. These papers, we trust, with the other articles which we have recently published on this subject, may lead the way to a better record in technical literature of the work of hydraulic power engineers.

Good progress is being made in the movement for uniform municipal statistics. A few weeks ago, the American Society of Municipal Improvements endorsed the form of summary of water-works statistics adopted years ago, and recently modified by the New England Water-Works Association. This form had already been approved and recommended for use by both the American and the Central States Water-Works associations. The American Society of Municipal Improvements has also adopted forms for statistics of streets and sewers. Last week, the American Gas Light Association adopted the report of its committee on Uniform Gas Accounting, as noted more fully elsewhere in this issue. The American Street Railway Association, and several associations of railway accountants, a few years ago, adopted a system of street railway accounting, and the National Electric Light Association has a committee on electric light accounting. While the gas, street railway and electric light committees were chiefly concerned with corporation accounting the reports of the several committees named form, or will form, a most important contribution to the broader work of making it possible to secure comparable statistics of all classes of municipal service, whether under public or private ownership. The New England water-works schedule, it may be added, is about to be sent out to all water-works officials who have heretofore used it, or who would like to use it hereafter, in making up summaries of statistics for their annual reports. All the figures thus compiled are, in turn, included in a grand summary in the "Journal of the New England Water-Works Association," of which Mr. C. W. Sherman, 1 Ashburton Place, Boston, Mass., is editor. Copies of the form mentioned may be secured by applying to Mr. Sherman. It is also to be hoped that city engineers and other officials in charge of public works will make use of the forms for streets and sewers adopted by the American Society of Municipal Improvements. Mr. Geo. W. Tillson, Municipal Building, Brooklyn, is secretary of that society, and Mr. C. C. Brown, Indianapolis, Ind., is chairman of the committee having the matter in charge.

We commented last week on President Vreeland's woful tale of the oppression of street railway corporations by municipalities, in his address before the Street Railway Association. So far as we have observed, the only sympathetic utterances which his sad story has evolved have appeared in the columns of certain class journals, which make it a matter of principle to favor the franchise corporations and oppose municipal control at every opportunity. The daily press unmercifully ridicules Mr. Vreeland's address. One of the Detroit papers remarked:

> The gentleman on the front seat who was sobbing violently while Mr. Vreeland described the persecution of the street railway companies is said to have been Mr. Jere C. Hutchins.

Mr. Hutchins, it should be explained is president of the Detroit United Street Ry. Co.!

The Detroit "Free Press" raps Mr. Vreeland's knuckles in an editorial which is so much to the point that we reprint a portion as follows:

> Mr. Vreeland made the most plausible case possible, but obviously he presented only one side.
>
> It is respectfully submitted that few of these corporations come into court with clean hands. For mercenary purposes and to obtain valuable franchises for less than they are worth, they have debauched common councils, legislative bodies and other official authorities.
>
> President Vreeland does not need any information as to how the surface railways in his own city of New York, and the municipal sources of gas and electricity are now in the hands of two great corporations behind which there is one alliance of men. The market value of their securities is over $400,000,000, while the plants could be replaced to-day for 25% of that amount. The people get nothing for this, except a service for which they pay sufficient to insure dividends on three parts water to one of honest investment. Even the original promoters have been largely frozen out, the gain going to the monopolists who worked the double leverage of "politics" and "financing."
>
> This does not tend to arouse the sympathy of the public that is thus mulcted under a wholesale plan of plunder.
>
> It cannot have escaped the attention of President Vreeland that they are sending men to the penitentiary from St. Louis for criminal manipulation in respect to franchises. Even he has not been so busy as to forget the scandal of corruption in Chicago when Yerkes' interests were seeking valuable concessions. He must know something of the abuses to which Detroit and Michigan have been subjected.
>
> It would be an ideal situation if there were that partnership between the electric lines and the people that Mr. Vreeland would now have exist. But an absolute requisite to a congenial partnership is honesty. By the process of bribery and debauchery the people who are now invited to the feast of brotherly love were first unmercifully plucked and then impudently requested to look pleasant.

In contrast with Mr. Vreeland's address were the remarks at another convention, that of the American Gas Light Association, held in New York last week and reported elsewhere in this issue. Mr. Wm. R. Beal, the President of the Association, in his address accepted public regulation of the gas industry as inevitable and said he would welcome the establishment of more State gas commissions, such as that which has for years adjusted the relations between cities and gas companies in Massachusetts. In one of the papers before the convention the principle of public regulation of gas rates was definitely accepted. The secretary of the society in discussing the paper advocated coöperation between companies and cities in working out a fair system of taxing and rate regulation.

All these things were said in a convention devoted to the interests of the gas companies. And they illustrate the attitude which progressive and intelligent men connected with public service corporations are rapidly assuming. The ostrich attitude which has been maintained in the past is not only undignified but it leaves the party taking it undefended against attack.

It sometimes pays in an unexpected way to contribute articles to Engineering News. More than a dozen years ago a piece of machinery was described and illustrated in this journal, the illustration being made from a working drawing furnished by the engineers then in charge. The enterprise in connection with which the machine was used soon after failed and the works were abandoned and have been untouched ever since. A short time ago new owners and new engineers undertook the completion of this long-abandoned work. The records and drawings in connection with the work had largely disappeared and the particular machine in question (which it was desired to duplicate) was so located as to be very difficult of access. Under these circumstances it occurred to some member of the office staff to look in the bound volumes of Engineering News, and the finding of the illustration above referred to saved the engineers in question a considerable amount of time and expense.

We may add that in other cases that have come to our knowledge where there had been no change of ownership or engineers, the records and draw-

ings published in Engineering News have been of great value for reference, where records belonging to the office had been lost or destroyed.

In discussing the New York Canal situation in our issue of Oct. 2, we urged that the Northwest would be glad to unite with New York for Government construction of a 21-ft. deep waterway from the Great Lakes to tidewater. The Minneapolis "Journal" comments favorably on our proposition, and says:

> It really does look as if, since a barge canal will cost so much, the time is ripe for the federal government and the state of New York to unite in a great ship canal work.

CONCERNING CONCRETE-FILLED COLUMNS.

Filling steel columns with concrete is not absolutely new, but it has been so seldom done and its advantages are so generally ignored by engineers that we are justified in calling particular attention to the valuable contribution in this issue from Capt. J. S. Sewell, Corps of Engineers, U. S. A., describing the concrete-filled columns in the new Government Printing Office at Washington.

Some two years ago, in editorial correspondence from Manchester, England, we described the concrete-filled steel columns which are used in the pier warehouses along the Manchester ship canal. Here there are special advantages in the concrete-filled column, for it can stand uninjured a side-blow from a heavy load swung from a crane in unloading vessels, which would cripple an ordinary column.

It appears to us, however, that no good reason exists why all steel columns used in building construction should not be filled solid with concrete. We heartily concur in the opinion expressed by Capt. Sewell that a concrete-filled column can have the weight of steel reduced enough to pay the cost of the concrete filling and would still be much stronger and stiffer than the steel column of the same cost without the concrete.

But strength and stiffness are not the only advantages secured by filling with concrete. Such a column would be far better able to resist fire. It may be said that columns in steel buildings are well enough protected by the fireproofing used; but we are speaking not alone of columns in fireproof buildings, but of columns in shop and mill construction, in factories, in railway stations, in warehouses, and a great variety of buildings for industrial purposes.

Actual fires have demonstrated that steel columns in such buildings will buckle and fail under the action of fire much sooner even than wooden posts. For proof of this statement we may refer the reader to our issue of July 2, 1896, in which the destruction of the Washburn & Moen wire factory at Worcester, Mass., was described and illustrated. This building had no wood of any consequence in its construction, except in the floors and roof, but it went down like a house of cards within a few minutes after the outbreak of a fire from an oil bath used for tempering springs.

We cite this $150,000 fire only as a typical example. Numerous similar fires have thoroughly proved that unprotected steel columns are highly vulnerable to flame, and that modern steel frame shop and factory buildings, which were at one time supposed to be practically safe from fire because built of incombustible materials, are little if any less liable to destruction by fire than the wooden frame buildings of a previous generation.

A concrete-filled column, on the other hand, would be, there is every reason to believe, as stable against fire as any construction known. This would be particularly the case if the concrete were made to not only fill but also envelop the whole column. In default of actual experience with such columns in fires, we may refer to the very numerous fire and load tests of floor constructions of concrete and concrete-steel. In view of the record which such materials have shown in fire tests when subjected to heavy bending stresses, one can feel assured of their satisfactory behavior when acting as a column.

And then there is the much-discussed matter of corrosion. We invited correspondence upon this topic in our issue of Aug. 28, and we are in entire agreement with the general tenor of such responses as have been received, to the effect that under ordinary circumstances corrosion in the interior of a closed steel column is not a danger to be feared. Nevertheless there are conditions under which such corrosion may take place. If moisture can reach the interior of the column, if steam or corrosive gases can gain access to it, if air can circulate through it even, there is more or less chance for corrosion. Under such circumstances filling the column with concrete recommends itself as an ideal method of protection.

Most engineers are already well convinced of the general efficacy of concrete coating as a protection to steel against corrosion. Additional and valuable data concerning this, however, is afforded by recent tests at the Insurance Engineering Experiment Station, which are described by Prof. Norton, the engineer in charge of the station, in an article published in this issue.

In former years, as we pointed out in our issue of Aug. 28, it was a favorite fad of engineers—we cannot dignify it by a better name—that an open column was better than a closed column, because its interior could be reached for painting and inspection. Accumulated experience has shown that thorough preservation by painting of even the exterior surfaces of an iron or steel structure is an expensive and difficult task, while the proper cleaning and painting of the interior of a latticed column is so much more troublesome and costly that the work is almost never thoroughly done.

Yet by filling the columns with concrete and making the mold large enough to include the exterior of the columns as well, engineers have at their disposal a means by which the painting of columns is rendered entirely unnecessary, not only at first but permanently. In fact a steel column concrete-filled should need no more care or inspection than a column of masonry. It may be added that from the point of view of appearance the concrete-filled (and covered) column would be far preferable to the steel column.

Another fact which structural engineers will appreciate is that very great flexibility in the design of columns and no small expense in their manufacture could be saved by the use of concrete filling. A steel column is made up of certain chief members—angles, channels or beams—connected by lacing strips or by thin cover-plates, the purpose of which is to cause the main members of the column to act together. Now, if the column is filled solid with concrete, the main members are much more effectively bound together than can be done by any riveted connection alone. Therefore the labor and expense on the steel connections can be reduced. Single lacing is as good as double lacing, for example, in a concrete-filled column. There are several styles of column which have been found economical to manufacture and convenient in erection, but which have been abandoned because of doubt as to their stiffness to resist eccentric loads or wind stresses. By adopting concrete filling, such columns might be made an entire success. Of course, it will be understood that in filling columns with concrete the wooden molds confining the concrete will naturally be put entirely outside the column, so that the lacing strips or other parts joining the main members will be wholly imbedded in the concrete.

Again, as Capt. Sewell points out near the close of his article, while the start will be to fill present steel columns with concrete, the development will almost certainly be toward the use of columns in which the concrete and not the steel will contribute the chief strength. It is of no small importance to the designer to have a substitute available to which he can turn when the exactions of trusts or the restrictions of labor organizations may make steel high-priced or difficult to procure. Cheap concrete is one of the certainties of the future. Dollar cement is among the probabilities. Is it not then important that the engineer has at hand a system by which he may reduce the quantity of steel necessary for his work without at the same time sacrificing the strength and safety of his structure?

Finally, as Capt. Sewell points out, experiments and tests on actual columns filled with concrete are highly desirable. We do not mean that an engineer need wait for such tests. Enough is known and enough has already been done in this field so that the use of such columns cannot be called experimental. Nevertheless, experiments and tests on actual columns would be of value to advertise the new departure and to convince that large class of men who will only learn by object lessons, and to supply data for the use of the designer.

At this time of the year several hundred young engineering students are searching for some really valuable and unworked field in which to conduct thesis investigations. We suggest to some of these ambitious boys that they undertake some tests on the comparative strength, stiffness or resistance to fire of steel columns filled with concrete and unfilled. There are so many different fields to be covered in such an investigation that a large number of students might undertake such investigations without their work overlapping.

Apropos of eccentric loading, we give space in this issue to two letters on that much-discussed subject. The best comment we can make on these letters is that concrete filling would provide safely for nine cases out of ten of eccentric loading without alteration in the steel work at all.

LETTERS TO THE EDITOR.

An Instance of the Lifting and Distortion of Piles by Ice.

Sir: During September of last year there was driven in Port Arthur, Ontario Harbor, a single row of piles enclosing a booming ground of some 30 acres for saw logs. These piles were driven about 10 ft. centers in a depth of from 5 to 14 ft. of water through a hard gravel and clay bottom to a penetration of from 7 to 12 ft. At the corners and openings for filling the boom, clusters of four and five piles were driven but not bolted together. Also in another part of the harbor piles were driven in 3 ft. of water through a hard sand to carry a temporary footbridge of some 300 ft. in length. After ice had formed in the fall all these piles commenced to rise and by February many of them were lifted from 7 to 10 ft., in some instances the ice alone holding them up. Those under the footbridge had raised and warped the platform to such an extent that it was unfit for use.

The above effects produced by the ice could not be attributed to excessive surging or wave motion as the harbor is well protected by a breakwater extending its full length, and furthermore the ice extended for several miles out from the shore. Further, the water level of Lake Superior lowers from 1 ft. to 15 ins. during winter months, hence the ice as a body would produce no lifting effect. The piles in question were tamarack of from 10 to 13 ins. diameter at the butt. The material into which they were driven was of too hard a nature for any local jarring to loosen them sufficiently for their own buoyancy to produce any lifting effect; hence it must have been an imperceptible movement in the ice which caused the trouble. Wharves in the harbor, that were built in some cases over 20 years ago, show no signs of heaving or warping, though in the spring masses of ice may be seen clinging to the piles at a height of 12 to 15 ins. above water level, showing the elevation where the ice formed the previous fall.

Possibly some of your readers have observed similar instances of piles heaving from ice action at other points on the lakes and can give an explanation as to the cause, also whether it is peculiar to Lake Superior alone, where a small tide effect may exist. I may state the thickness of the ice in this instance was from 24 to 30 ins.

Yours truly, J. Chalmers,
Assistant Engineer, Canada Northern R. R.
Winnipeg, Manitoba, Oct. 11, 1902.

Letting Contracts for Grading Without Classification.

Sir: In a recent number of Engineering News, the specifications of the Charleston dry dock were severely commented on editorially. In continuance of this same subject, I would like to call your attention to a practice lately inaugurated by a number of our large railway companies, and that is of letting grading without classification. That is to say, instead of a contractor bidding separately for earth, loose rock, solid rock, etc., as the case may be, he makes one price for the material, just as it may be found.

This seems to me a most reprehensible practice. Take a long line of railroad, with millions of cubic yards of excavation, varying in character, it seems to me to be a veritable guess on the part of the contractor to arrive at an average price. The relative quantities of each kind must certainly be known to arrive at the price intelligently, and if the railroad company's engineer does not possess this information, how is the contractor to obtain it? Perhaps by wearily plodding over the line taking notes of the various geological formations. Is

this not the business of the locating engineer, while running the preliminary lines and making the final location? While the surveys are in progress, ample time is available to do this, if not by making borings, then by an actual examination of the ground, out-croppings, adjacent cliffs, surface indications, etc.

I have located a few miles of railway in my time, and it always was the duty of myself and assistant to take copious notes of the formations upon which to base a proper classification of the material likely to be encountered.

A case in point: Some years ago a prominent railway company let some heavy construction work. On one part of the line was a very heavy, long cut. It was popularly supposed that the material it contained would be solid rock. One of the bidders, formerly a civil engineer, had his doubts about this, and on his own account had some borings made, which confirmed his belief, the material being other than rock. He made a bid somewhat lower than the usual bids for rock, and, of course, got the work. On account of his cleverness he was able to clear about $50,000 on this cut alone. Had the railway company invested about $100 in borings, it might have saved the bulk of this sum.

Railroad Engineer.

San Francisco, Oct. 6, 1902.

Objections to Electing City Engineers by Popular Vote.

Sir: The workings of a nominating convention held here last week possess features that bear on the question of selecting city engineers by popular vote, discussed in your editorial note of July 24, and in Mr. Steece's letter in your issue of Aug. 7.

Election by the people is theoretically one thing, and practically something else. It is particularly so when, as is almost always the case, men for several different offices are nominated by the convention. These offices are not of equal value, and those of greater value overshadow those of less. I am not speaking of real value, but of the value attached by the popular mind as reflected by a nominating convention. The voters at large concern themselves very little with the office of city engineer when considering along with it the offices of mayor, city treasurer, alderman-at-large, and others of supposed importance, peculiar to each particular city. Therefore it generally comes about that a man with a pleasing exterior and some good hustling friends has no trouble in getting the nomination, while men less favored in these two respects, but perhaps far superior in ability, are passed over.

Appointments by mayors and city councils are not always based on qualifications; but are nominations by party conventions less open to this criticism? Mr. Steece says that mayors and members of city councils rely on politicians to name the officers; but on whom do the delegates rely? If the politicians can get the appointment from the mayor, it is pretty safe to say that they can name their man in the convention. Mr. Steece says "there is no reason why a good engineer should not be a fair politician." Nine times out of ten he is not, and that for the same cause, whatever it may be, that keeps out of active political work the thousands in our cities who, by education or ripe business experience, are best fitted to lift municipal affairs to a plane far above the one they now occupy.

A convention well enough informed to choose the right man for city engineer can be depended upon to nominate mayor and council of proper caliber to make a good choice, and a mayor's chances of knowing the qualifications of engineers are superior to those of conventions at large.

The matter of influence exerted on the engineer is dependent on the quality of the administration, and reverts thus to the nominating convention, our primal source in both methods of procedure.

Yours truly, G. M. B.

Salt Lake City, Utah, Oct. 9, 1902.

Measuring Load of a Vessel by Gaging the Draft.

Sir: It is desired to measure (weigh) the superincumbent load of a deck scow by displacement, and it is intended to do this by means of gages to get the difference between light and loaded readings. The question is whether it is better to use two or three gages. If three, should the average be taken, or should the middle reading be doubled and all divided by four? Also what is the proper position of the end gages? It is likely that the method of loading the scow would influence the gages at the end, tending to raise or to depress them, making a material difference in the displacement as determined from the readings. I should be glad to hear from some of your readers on this subject.

Water Gage.

New York, Oct. 3, 1902.

(The problem here presented differs in no way from the problem of estimating earth yardage in railroad work by the averaging end areas method. If the gages are placed equal distances apart, then the "average gaging" would be

$$\frac{\frac{a}{2} + \Sigma m + \frac{b}{2}}{n}$$

in which

a = gage reading at one end of scow.
b = gage reading at other end of scow.
Σ m = sum of all intermediate gage readings.
n = total number of gages.

If, due to inequality of loading, the scow itself becomes warped materially, we know of no simple way of estimating the displacement; but it would seem possible to avoid such unequal loading. The end gages should be located at the intersection of the curved or beveled nose of the scow, with the horizontal bottom. We have in the foregoing assumed a rectangular scow cross-section; if, however, the cross-section is curved, the gage reading can with exactness be applied only to the particular section at which the gage is located.—Ed.)

Railroading and Other Things in France.

Sir: A curious example of French railroading came under my notice recently at Culoz, France while on a journey from Paris to Switzerland. The place is an important junction and it was necessary to switch a car out of our train and put it on a branch line track. Part of the switching was done by a horse and part by a diminutive switching engine—each motive power, by the way, having to be hunted up from somewhere by a special messenger while we waited. The baggage to be taken on was all piled into the car before the baggage to be left was removed. Altogether the stop required 20 minutes. A similar amount of work would be done at any third-rate junction in the United States in less than five minutes.

Coal is brought in bulk to Paris in canal boats. There it is loaded into bags of about 100 lbs. capacity. A man carries up a bag at a time and lands it on a stand about as high from the ground as the body of the two-wheel cart, which hauls the coal away. The man has to climb a ladder from the bottom of the boat to the deck, then a second ladder from the deck of the boat to the quay. The bag hangs from his head and rests on the man's back, leaving both hands free. More than a week is needed to unload a boat which carries about 350 to 400 tons. Grain and other articles carried in bags are unloaded in the same primitive fashion.

In the construction of important buildings very large stones are often used. Derricks, such as we use at home, are not known here for building purposes. In fact, I doubt greatly whether the police authorities would allow such rank heresy as a machine which would in one operation take a stone from a wagon and land it in its proper place in the wall. No, they put up here a square tower just outside of and rather higher than the building to be erected. The stones are hoisted generally by hand to the height where needed. There they are landed on the wall and gradually moved to the position they are to occupy, which may be several yards away from the first landing place.

From what I hear, and judging by what I have seen by the work of the painters who have been occupied at cleaning the house in which I have my apartment, I should say that out of 11 nominal working hours, the actual working time is about 5 hours. Men come to work somewhere between 6 and 7 a. m., according to season; they quit at about 9 to go and take a drink, at which operation from half an hour to an hour is spent. At 11 o'clock they go for their dinner, a meal which requires an hour and a half to two hours, including the nap afterwards. At half-past three or four they go off for the goûter, as it is called, at which they spend at least a half-hour more, and after which they continue until knocking-off time, generally about 6 p. m. When they are at their places anything serves as an excuse to stop work.

Some work is exceedingly good, other is equally bad. I have seen buildings of the real Buddensieck type, and from time to time I read of the collapse of a building or of a block.

Per contra, I saw in the Canton of Geneva the other day a very neat system of running standard gage cars over narrow-gage tracks. There are in this canton many country roads on which trolley lines are run, carrying freight as well as passengers. The usual type of freight car in Europe runs on four wheels. Small four-wheel trucks, the wheels about 12 to 15 ins. in diameter, are made to suit the gage of the trolley lines. These trucks carry on each side a sort of saddle which is above and outside the wheels of the truck; the median plane of the saddle is vertical and midway between the two axles of the truck. This saddle receives the wheels of the freight car to be carried. When a car is to be moved a truck is placed under each pair of wheels; the car is then ready to be hauled away. I have seen cars loaded with coal at a distance of a mile or more from the nearest railway in trains of three or four cars, each mounted on trucks, hauled by a small dummy engine. In like manner goods in bulk can be shipped from a factory remote from the railway line.

F. A. Mah...

Paris, Sept. 30, 1902.

The Brooming and Breaking of Piles by Hard Driving.

Sir: Having read with interest the experience of other engineers in driving piles it may be of interest to them if I record briefly some experiments and observations of my own on certain phases of such work. Being young in the business and not being able to get the necessary information from technical papers and books that I needed before getting up the plans and specifications for driving piles for two factory buildings, I decided to make a few tests.

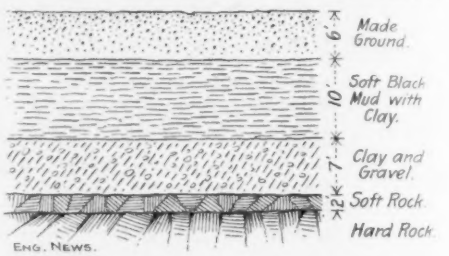


Fig. 1. Sketch of Materials Penetrated in Making Test.

The ground was "made ground," filled in about 6 ft., but before filling it was covered with water at high tide. The materials we had to penetrate are indicated by Fig. 1. I drove five piles about 4 ft. apart, and then pulled them out. The piles were straight yellow pine, 12-in. butts, 6 in. points and 26 ft. long. Pile Nos. 1 and 2 were driven with heavy blows, about 18 to 24 ft. drop (2,800-lb. hammer), and until the last four blows they penetrated only 1½ ins. Pile No. 3 was driven with 10-ft. blows until the penetration was 1½ ins., in the last four blows. Pile No. 4 was driven with 20-ft. blows until it reached the soft rock; then, with 10-ft. blows until the penetration was 1½ ins. in the last three blows. Pile No. 5 was driven with 20-ft. to 24-ft. blows until it reached the soft rock, then with 12-ft. blows until the penetration was 1½ ins. in the last three blows.

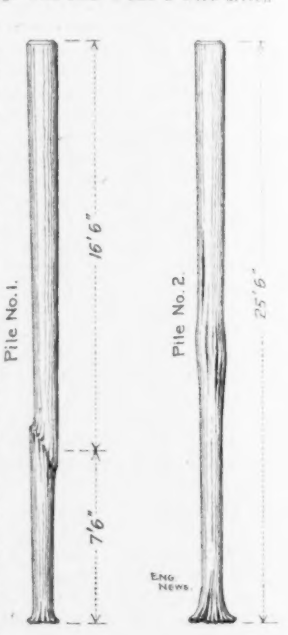


Fig. 2. Sketch of Piles No. 1 and No. 2 After Pulling.

Upon pulling these piles we found that pile No. 1 had broken about 8 ft. from the end; (Fig. 2a) pile No. 2 had split badly in center and the bottom looked like an inverted mushroom (Fig. 2b); piles Nos. 3 and 4 were unharmed, and pile No. 5 was unharmed, but showed the effect of faster driving than Nos. 3 or 4.

In the specifications I made a note that during the last 5 ft. of penetration the pile must not be struck with more than 12-ft. blows and until the pile penetrated only 1½ ins. under the last three blows.

Since being employed in my present position I have crossed the new Grey's Ferry Bridge daily, and about four years ago, when it was being built I watched the pile drivers driving the piles for the cribbing and fenders and observed that they struck them with anything from an 8-ft. to 16-ft. blow. The piles were 55 to 60 ft. long, 15-in. butts and 4 to 6-in. points. In June of this year the Pennsylvania R. R. pulled out a hundred or more of these piles to make room for a new railway bridge. The piles are now laying on the north bank between the highway and railway bridge; some are badly broomed on the point, others are broken off, and others are unharmed. I think it is reasonable to suppose that the piles that were struck with a 15 or 16-ft. blow (3,000-lb. hammer) are the ones that are broomed or broken. Hoping this information will be of value to some of your readers, I am

Yours truly, E. Winfield Finkbiner,
Assistant Engineer, Harrison Bros. & Co.

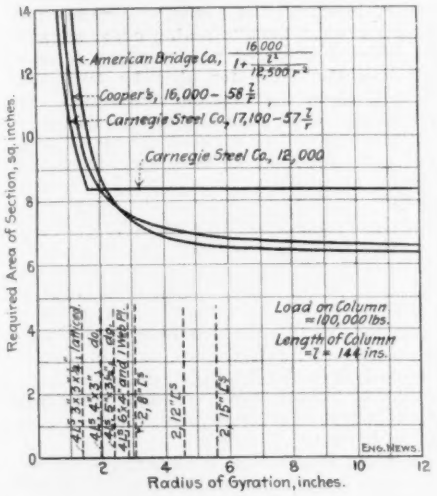
Philadelphia, Pa., Oct. 13, 1902.

I-Columns Preferable to Box Columns for the Framework of High Buildings.

Sir: Referring to the editorial in your issue of August 28 on the subject of box columns in office building construction, I desire to call attention to a few facts bearing on the subject and tending to show that the box section is not used exclusively for this purpose, and is not in fact as good a section, all things considered, as the I-section made up of latticed angles or plates and angles.

During the past year six large "skyscrapers" have been or are being built in Pittsburg, namely, the Frick Building, the Union Station, the Arrott Building, the People's Savings Bank Building, the Keystone Bank Building and the Farmers' Deposit Bank Building. Of these, the first two have I-section columns, and the balance have box columns of channels and plates. The Frick Building is the only one having granite curtain walls for its entire height of 22 stories, the others being of brick or terra cotta above the first few stories; it is consequently the heaviest of the group, as far as loads on the columns are concerned. So much for the practice around Pittsburg.

Now as to the claims of special merit for the box section on the theoretical grounds that the material is better distributed about the neutral axis, I beg to call attention to the fact that while this is true of long columns generally, yet in the present instance the very heavy loads sustained, combined with a comparatively short unsupported length (only about 12 ft. in most cases), tend to eliminate the special advantages of any one section on this score by bringing about a condition of direct compression in the columns, in place of the theoretical long column conditions. To bring this out clearly I have prepared the accompanying diagram, in which vertical ordinates to the


American Bridge Co., 16,000 / (1 + l²/12,500r²)
Cooper's, 16,000 − 58 l/r
Carnegie Steel Co., 17,100 − 57 l/r
Carnegie Steel Co., 12,000
Load on Column = 100,000 lbs.
Length of Column = l = 144 ins.
Required Area of Section, sq. inches.
Radius of Gyration, inches.
ENG. NEWS.

three curves shown represent the sectional areas required for a column of constant length of 12 ft., under a constant load of 100,000 lbs., but with varying radii of gyration, using unit stresses as determined by three well-known compression formulas, noted on the diagram. From the shape of these curves it will be seen that if the radius of gyration is sufficiently large, any considerable variation in radius can occur without materially affecting the amount of section area required. The Carnegie Steel Co. has recognized this and use a constant unit stress for all cases where the radius exceeds certain limits, which limits include practically all columns of office buildings, thereby simplifying the operation of designing, and giving practically as good results as any other method; especially when it is considered how much approximation is done in determining the column loads in the first place.

For the first two and three stories of columns at top of the building, the loads do not admit of using economical I-sections, but when it is considered that the I-section has one system of latticing, while the corresponding box section has two; also that practical considerations fix the minimum size of channels that can be used for this purpose at about 8 ins.; also that these columns form a small part of the total material; it will be seen that the box section even in these stories has but a small margin of advantage in actual material. As we descend, the column loads increase rapidly, requiring sufficient metal to use an economical radius of gyration; at the same time the difference between the radius of the I-section and that of the corresponding box section becomes less and less.

The effect of eccentric loads on the upper stories of columns is greater on the I-section in one direction than on the box section, but by proper care in designing, this can be reduced to such limits as not to affect the section, in most cases, and provided for directly in others, without affecting, to any appreciable extent, the total amount of material required.

The I-section admits also of more efficient wind portals, especially at the top of the building, where effect of wind is a greater percentage of the total load than lower down. In these upper stories the sections are considerably stiffer in one direction than the other, and by selecting carefully the location for portals in the two directions, placing the columns to properly act with the girders between them to complete the portal effect, the building can be given considerable stiffness without any increase in section of columns; but the portals, to be effective, must be efficiently connected to the columns by brackets—indeed the connection of the brackets to the columns will, in most cases, be the critical point in determining the efficiency of the portal—and the I-section column offers the least obstruction to making these connections efficient.

Since the claims for theoretical advantages are shown to be slight, if any, there is little to commend the box section over the I-section; for, practical considerations are all in favor of the latter. In the drafting room, shop and erection work innumerable points may be cited where the cost is increased and the rigidity of the structure lessened by the complex and clumsy details which are made necessary by use of the box section. The box section made up of two channels and cover plates must be bolted up and riveted together in four parts; then these four parts fitted up again, and the whole riveted up to make the complete column; this makes the shop cost expensive and brings the rivet heads of many connections inside, making it impossible to correct mistakes at any time in the future.

Compared with this, the I-section column is bolted up complete, and then riveted, is one or two operations against five in the former case, and all details are open to view, permitting changes to be made at any future time to correct errors or make additions.

One factor that often enters and decides in favor of the box section is the relative quickness of delivery of the various sections from the mills. This is usually a matter of great importance in building construction of this kind, and when channels are more easily obtained than angles, channel columns will generally be decided upon regardless of other considerations.

Yours respectfully, Chas. Worthington.

Pittsburg, Pa., Oct. 11, 1902.

The Need of Established Standards for Turbine Measurement.

Sir: Engineering News of September 18, 1902, contained a highly interesting article, entitled "Notes on the History of Turbine Development in America." It is clearly shown that the high-power turbine of America is a United States development; also that the first ideas were borrowed from France, and that the several turbine inventors have added to and borrowed from each other in its evolution, until its attainments have culminated in the high figures shown in the author's tables.

Observation has led many to believe that the American turbine water wheel has not received the special interest from scientific men that it deserves, therefore no definite bases have ever been established from which uniform and logical reasoning can be followed in describing it. The lack of fixed starting points in the measurements of the turbine causes so much confusion in the minds of purchasers that it seems reasonable and right to suggest a movement that will in some manner lead to an intelligent standardization.

The development of turbine plants since the advent of electrical transmission has been so marked that the engineering fraternity should become interested enough to make some effort to correct these faults, as has been done in other engineering lines. It is due to the buyers of turbines that some common system of measurements and phraseology be agreed upon and adopted by the manufacturers of turbines, enabling all to understand and make comparisons when discussing the relative merits of turbine water wheels.

In order to make this clear, the comparisons made by the author of the paper in Engineering News of Sept. 18 will be used to show how it is possible to mislead when misleading is not intended. It is evident that the author intends to convince the reader that "Rice's Victor" is a superior machine to the "Victor," which is shown to be superior to all others. His line of reasoning is well calculated to establish this point in the mind of the purchaser, and no doubt many of the other turbine men can show equally well that their several turbines should head this list of reliable high-power turbines. This cannot be laid to the several turbine builders so much as to the system or rather to the lack of system in vogue.

This lack of system comes from developments carried on, as the author has stated, by the "cut and try plan." Our turbine inventors have not all been trained engineers. Each one has used his own terms when referring to the several parts of his device; each one has chosen his own methods of measurements, and each one quotes tests most favorable to his turbine.

It can be shown that the results of these tests vary from nearly zero to 200% efficiency, according to the intelligence of the inventor, and sometimes the merit of the turbine. But out of this random endeavor much good has come. The American turbine, while not perfect, is practically developed, and some system should be adopted that will elevate it and its manufacturers to a proper plane from an engineering standpoint. Brother engineers across the ocean point their fingers righteously at the American turbine and its manufacturers, and the article in Engineering News will hardly convince them of the facts its author intends to convey.

Only a few references will be made to the author's statements. On p. 209, last column, in the third paragraph, we read:

> A 15-in. wheel when tested and found to give more power and efficiency than the Hercules was named "Victor." This 15-in. wheel was tested by Emerson, March 26, 1878, and gave 91.39% efficiency under 18-ft. head, 30.12 HP., 355 revolutions. It was then taken out and tested some weeks later, giving 91.41% efficiency under 17.94-ft. head, 30.40 HP., 352 revolutions. These results were so remarkable that this wheel was placarded and consigned to the storehouse where it can be found to-day.

The author did not state that owing to a laxity in methods some of Mr. Emerson's tests are open to criticism and doubt. There are some intelligent engineers who believe that this same turbine cannot repeat this performance in a reliable test.

On p. 210 a very interesting table appears showing the advance of the horizontal turbine as manufactured by one concern; this looks well to the layman and would indicate somewhat the lion's share of horizontal turbines as being built by this one company. One interesting fact has been omitted. It is known that many of these turbines are credited with from 10% to 25% less power than they were intended to develop under the conditions. It would seem that this fact should not have been omitted in an article following so closely the history and development of the American turbine water wheel.

In the same column are two comparison tables that are so much in contrast to the previous reports of tests that they can hardly be passed by. In the previously cited tests the actual heads, revolutions and horse-powers were given, but in these nothing is given but estimates based on the tests. The formula commonly used in these reductions has been questioned by good authority, hence the reader is left slightly at sea. If the actual figures of the tests were given, the reader would have the satisfaction of making his own comparisons.

In the last table, which appears to be a conclusive comparison, there is the greatest possibility of misapprehension. The author compares the powers of five 30-in. diameter turbines under 20-ft. head. Since the testing flumes of the Holyoke Water Power Co. are recognized as official in America, it must be assumed that these tests were made at Holyoke. Since it is hardly possible to secure a 20-ft. head in the flumes, we must assume that the catalogued powers, etc., for 20-ft. head are estimated. This cannot in itself mislead badly in the comparison, as the same formulas were no doubt used in all cases; but there is a possibility of the reader getting an erroneous idea by not being familiar with the manner in which the different wheel makers determine the diameters of their turbines. Some observation in this line has led the writer to believe that each wheel maker has his own system; and that these systems are so variable that a competent mechanic unfamiliar with the turbine cannot tell its size or diameter by the most careful measurements. Some manufacturers use the greatest diameter in inches when determining the size. Some use the smallest diameter. Some measure the diameter on a conventional radius, and others estimate the diameter on the relative peripheral velocity to spouting velocity due to the head. This makes it possible for the sizes alluded to, when taken from catalogues, to vary from 30 ins. to 45 ins. diameter (if all were measured in the same manner), making a difference of over 100% in area of wheel diameter. This in turn will undoubtedly make a difference in the powers of the several 30-in. diameter turbines when compared for the same head.

It can readily be seen that such comparisons will result in confusion where no common system of measurement is used in the turbines compared. In illustration of this the writer will mention a development and test made from a turbine standing low in the author's list. The manufacturer of this turbine claims to rate the sizes of his turbines by their speed or relative number of revolutions to spouting velocity due to head. This is a rational method as there is a fixed starting point.

A set of turbine patterns was manipulated until it produced a turbine, the revolutions of which were those of a theoretic 28-in. standard turbine of his own make. The writer saw this turbine tested and his notes are now before him. They show 169 HP. under 17.11 ft. head and 190.75 r. p. m. This, reduced to 20 ft. head as per the author's list, will show 211 HP. for the theoretic 28-in. turbine. The highest efficiency approximated 84%. If a turbine of equal merit were constructed 30 ins. in diameter, being the size of "Rice's Victor," the power shown would be 243 HP. (assuming that the power increases as the squares of the diameters, which is approximately correct). This is approximately 33% greater power for a 30-in. diameter turbine than "Rice's Victor," and more than 100% greater than that shown in the table for the Hercules turbine.

Please understand that this letter is not for the purpose of questioning the author's deductions, but its intention is to show the great need of a reliable system of measurements and terms to be used in dealing with a class of machinery that merits the attention of our best

engineering talent. This need is so great that it would seem that some one or all of our interested engineering bodies would start a movement that will result in formulating and establishing a definite system whereby some relative ideas of turbines can be formed by the dimensions given them. The buyer of turbines who is somewhat acquainted with the phraseology of steam engineering is often mystified by the manifold and unsatisfactory comparisons made without intent to deceive. Trusting that the Engineering News will give this matter the consideration it deserves, I remain, yours truly,

Mark A. Replogle, M. Am. Soc. M. E.

121 Aqueduct St., Akron, O., Sept. 30, 1902.

Structural Columns Under Eccentric Loads.

Sir: The term of eccentric loading does not occur in Gordon's formula, but in building construction this term must not be omitted. The metal of one length of column should be directly over the metal of the length below, through the entire height of the building; this necessitates the application of loads to the sides of the column, and unless the loads are equal and on opposite sides of the column, the effect is to increase the stress on the side on which the greater load is applied. The concave side of a bent column is subjected to three kinds of compressive stress—that due to centric loading (uniformly distributed over the entire section), that due to the eccentric position of the load, and that due to the bending of the column.

The stress, p_1, on the extreme fiber due to centric loading and flexure of column, under loading equivalent to a uniformly distributed stress p, we find from Gordon's formula

$$p = \frac{p_1}{1 + a \cdot \left(\frac{l}{r}\right)^2} \text{ to be } p_1 = p\left\{1 + a \cdot \left(\frac{l}{r}\right)^2\right\}.$$

The stress p_2 due to the eccentric position of the load, is given by

$$p_2 = \frac{M}{R};$$

or, since $M = P \cdot d$,

$$p_2 = \frac{I \cdot d \cdot y}{I} = \frac{P \cdot d \cdot y}{A \cdot r^2} = \frac{p \cdot d \cdot y}{r^2};$$

where

M = bending moment, caused by the eccentricity of the load P; M = P d;
d = distance of eccentric load from axis of column;
y = distance of extreme fiber from center of gravity of the section;
r = radius of gyration of the cross-section;
R = moment of resistance of the cross-section;
I = moment of inertia of the cross-section.

We may now write, as the total stress on the extreme fiber at the elastic limit, due to compressive stresses from above mentioned three causes—centric loading, eccentric loading and flexure of column:

$$p_1 + p_2 = f = p\left\{1 + a \cdot \left(\frac{l}{r}\right)^2\right\} + \frac{p \cdot d \cdot y}{r^2},$$

from which we obtain,

$$p = \frac{f}{1 + a \cdot \left(\frac{l}{r}\right)^2 + \frac{d \cdot y}{r^2}};$$

where

f = elastic limit of the material in compression.

For calculation of the required cross-section of columns under combined centric and eccentric loading, proceed as follows: (1) Find the cross-section required for the total load, both centric and eccentric combined, the whole considered as applied centrally; a column formula, as Gordon's, should be used for this to allow for flexure. (2) Find the required area to resist the bending moment caused by the eccentricity of the load using as values of r, the radius of gyration, and y, the value of these quantities in the section assumed from (1). Then, (3) if both areas can be added together without changing the radius of gyration and y materially, the sum is the required cross-section. But (4) if, in providing for the new area, the radius of gyration and y are changed materially, then assume a new sectional area with corresponding r and y, and proceed as before.

In building construction the value of the term $a\left(\frac{l}{r}\right)^2$ is usually so small, due to the ordinarily short length of columns in buildings and to the fact that the ends are flat bearing, that the second term in the denominator—in the equation

$$p = \frac{f}{1 + a \cdot \left(\frac{l}{r}\right)^2 + \frac{d \cdot y}{r^2}};$$

—is of little importance. Therefore, in designing short columns with eccentric loads, as used in buildings, the bending of the column may be neglected. We then have for combined concentric and eccentric loads:

$$p = \frac{P}{A} \pm \frac{M}{R}, \quad \text{or,} \quad p = \frac{P}{A} \pm \frac{P_2 \cdot d \cdot y}{A \cdot r^2};$$

$$A = \frac{P}{p} \pm \frac{P_2 \cdot d \cdot y}{p \cdot r^2};$$

where

A = area of cross-section of column;
P = total load on column (both eccentric and concentric);
p = maximum permissible stress per square inch;
M = P_2 d = bending moment caused by eccentric load P_2;
d = distance of eccentric load from axis of column;
y = distance of extreme fiber from center of gravity of section;
r = radius of gyration of cross-section;
R = moment of resistance of cross-section;
I = moment of inertia of cross-section.

Krs.

St. Louis, Mo., Sept. 25, 1902.

Sir: I give here an answer to the question of F. D. B. in your issue of Sept. 25, concerning eccentric loads on building columns.

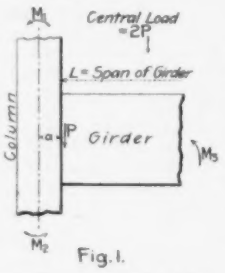


Fig. 1.

Fig. 1 is a type of such a connection. The girder is riveted to one side of the column, and the end reaction P is applied to the column at the center of the bearing of the rivets in the column metal, at a distance a from the center of gravity of the column. A moment P a is thus induced. This moment is resisted by three forces acting at the three connections adjacent to this one, viz.: The column connection at the floor above, the column connection at the floor below, and the connection at the other end of the girder, each causing the member connected to it to act as a beam, and thus setting up three resisting moments, M_1, M_2 and M_3, as shown in the figure. The ratio between these forces is determined by the relative stiffness of the arms, or beams, on which they act. It is apparent that the girder, which is designed to resist a moment of P into perhaps half its span L, will be stiff enough to resist without any appreciable increase in deflection the entire additional moment of P a, since a is only a few inches. This resistance, however, cannot be offered until the column has been inclined from the vertical, by an angle equal to the angle the end of the girder has deflected from the horizontal under the load 2 P. When this has taken place we may assume that the girder resists the entire remaining moment. Consequently, the moment due to eccentricity from a deep girder with an almost inappreciable deflection is of small consequence, but should the same load be applied by a shallow box girder, the eccentric effect on the column might be considerable. It is interesting to note that the resisting moment in the girder sets up a tension in the bottom rivets of the connection, and that no resistance could be expected from a girder resting on a shelf.

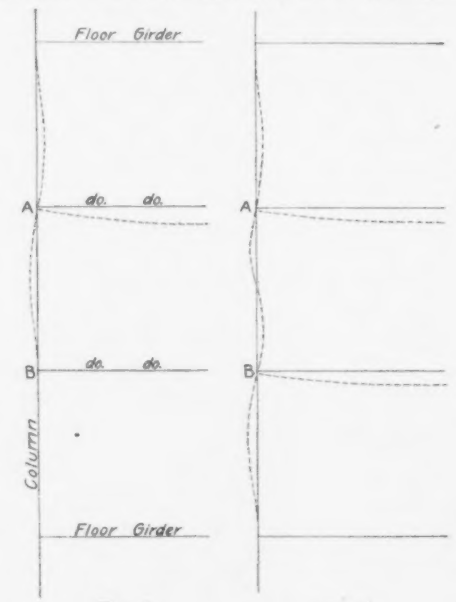


Fig. 2. Fig. 3.

Let us suppose that the column extends through several stories, and has eccentric connections on each floor, as is shown in Fig. 2. Such a case would be a column in an outside wall. In order to show the effect more clearly, the center lines only of the columns and girders are indicated, each girder is supposed to be attached rigidly at right angles to the column. Now, suppose the girder at A to be loaded, causing it to deflect and the column to bend, as shown by the dotted lines. It is plain that the point B cannot move horizontally as it is held in position by the girder, and that the column will assume a reversed curve as indicated, and will tend to lift up the far end of the girder at B. The resistance to this uplift will prevent the moment and consequent curvature from being felt further down the column.

If now the girder at B is also loaded, as is shown in Fig. 3, the point of contraflexure will be moved upward to the center of the distance between the stories, while the points through which the column is bent will remain the same. This means that the stresses due to the deflection of the girder will be increased somewhat, yet in no case can they be doubled, because the distance between the points of contraflexure cannot be reduced by one-half. This examination shows us that the deflection of the girder attaching to the column, and not the eccentric loading, is the important consideration.

In most building work a column formula is used which for ordinary ratios of $\frac{l}{r}$ gives an allowable unit stress of from 13,000 to 15,000 lbs. It is customary, however, to avoid going above 12,000 lbs. in any case, and the remaining 1,000 or 2,000 lbs. may be relied on in the majority of cases to cover the effect of eccentric loading. Exceptional cases will require special consideration.

Clarence W. Noble.

2315 N. Paulina St., Chicago, Oct. 3, 1902.

THE CAUSE OF MASONRY DISINTEGRATION.

By Halbert Powers Gillette.*

Masonry in northern latitudes disintegrates far more rapidly than it does nearer the equator; and freezing of water in the pores of the mortar or of the stone itself is commonly given as the reason of this rapid disintegration in cold climates.

The Egyptian obelisk that for centuries withstood Nature's attacks while on its native soil, began to spall off rapidly when brought to America and placed in Central Park. Why? Prof. John S. Newberry, of Columbia, answered with the one word, "Frost." Acting under his advice the obelisk was coated with paraffine to prevent the absorption of water, that by freezing was supposed to be the cause of destruction. From time to time since then the writer has inspected the obelisk, and has noted that new spalls continue to be found at its base; and this fact first led him to question the soundness of the frost theory.

If frost is not the active agent of destruction, it follows logically that alternate expansion and contraction of masonry due to temperature variations must be the primary cause of disintegration. In this paper will be given the reasons for believing that the phenomenon of masonry destruction in cold climates is due primarily to stresses developed by expansion and contraction due to changes in temperature. We shall first consider briefly a few phenomena that the frost theory fails to explain.

According to the frost theory, the most porous masonry materials should be the least durable. Turning to Baker's "Masonry," we find the following:

	Per cent. of water absorption.		Per cent. of water absorption
Granite	0.13	Sandstone	4.1
Marble	0.33	Brick	10.0
Limestone	2.6	Mortar	25.0

According to the foregoing, concrete, which is 40% mortar, and brick should disintegrate far more rapidly than natural stone masonry if frost is the active agent.

On the contrary, the writer has frequently observed that brick houses built upon stone foundations show a much greater durability of the brick part than of the stone. Portland cement concrete has within recent years come to be recognized by engineers as being more durable than stone masonry. The writer has also observed that in long masonry retaining walls the mortar in vertical joints, especially near the ends of the wall, is the first to crumble; while, according to the frost theory, the bed joints should be the first to fail, since they afford a better lodgment for water.

We have therefore at least three facts incompatible with the frost theory, namely, (1) that granite and other stone continue to crumble after closing the pores with paraffine; (2) that brick and concrete masonry, although many times more porous than stone masonry, are most durable, despite their lower unit strength; (3) that bed joints of mortar should fail first, but, in fact, they do not.

It will next be shown that all phenomena like those above cited may be fully explained by a heat expansion theory.

In the first place, variations in temperature are

*791 Powers Block, Rochester, N. Y.

far more sudden and cover a much greater range in northern than in southern climates, which accounts for the rapid failure of such stone as the Egyptian obelisk when moved northward.

To facilitate comparison, the writer has tabulated the coefficients of heat expansion of different materials; and in the accompanying table, in column C, will be found the coefficient of linear heat expansion of a block of material a foot long, for each degree Fahrenheit, according to the authority cited.

Material.	Authority.	C: coefficient of expansion.	E: modulus of elasticity.	f for 100°.
Brick, common	Haswell	.0000012	3,500,000	420
" fire	Haswell	.0000028	.....	...
Cement(?)	Haswell	.0000080	.....	...
Concrete (1:2:4):				
Lehigh Portlnd and limestone	Pence*	.0000055	1,000,000	555
			2,000,000	1,100
Lehigh Portlnd and gravel	Pence*	.0000054	.....	...
Glass	Haswell	.0000050	5,500,000	2,750
Granite:				
Aberdeen	Dana	.0000044	.....	...
New England†	Dana	.0000048	5,500,000	2,640
			13,000,000	6,240
Ice	Ganot	.0000028	335,000	...
Iron or steel av'g.	Haswell	.0000066	27,000,000	17,620
Limestone:				
Sing Sing, N. Y.	Dana	.0000057	.....	...
Bedford, Ill.	Pence*	.0000056	1,500,000	940
			3,300,000	1,850
Marble	Ganot and Haswell	.0000048	2,500,000	1,200
Sicily	Dana	.0000061	.....	...
Pottery:				
Wedgewood	Enc. Brit.	.0000049	.....	...
Bayeux	" "	.0000092	.....	...
Quartz, along axis	" "	.0000028	.....	...
Perp. to axis	" "	.0000043	.....	...
Sandstone	Haswell	.0000068	.....	...
"	Haswell	.0000108	.....	...
"	Ganot	.0000095	.....	...
" red§	Dana	.0000096	.....	...
" "	Thurston	.0000033	.....	...
" "	Thurston	.0000055	.....	...
Slate	Spons Dic.	.0000055	7,000,000	3,850
Wood, white pine		.0000023	1,800,000	410

*Prof. W. D. Pence in a valuable paper on "The Coefficient of Expansion of Concrete," read in 1901 before the Western Society of Engineers, gives results of tests.
†Fine grained. §Sandstone, red Portland, Connecticut.

E is the modulus of elasticity in pounds per square inch; f is the pressure in pounds per square inch at the ends of a column when restrained from expanding under a rise of 100° F.; f is obtained by applying the well-known formula

$$f = \frac{E\,S}{L}, \qquad (1)$$

in which

S = the total elongation.
L = the length of the column.

In this case, $S = 100\,C\,L$, whence

$$f = 100\,E\,C. \qquad (2)$$

It will be observed that the pressure f is entirely independent of the length L of the column, and varies only with E and C, so that it matters not whether a wall be 10 ft. or 100 ft. long, the pressure developed in the vertical joints if expansion were absolutely prevented would be the same.

A uniform rise of temperature in a wall cannot produce any increased compressive stress in the mortar of the horizontal joints, because there is nothing to resist vertical expansion.

Expansion, however, is never entirely prevented throughout the wall, unless, as in a dam, the ends of the masonry abut against solid bed-rock; but in every wall the friction of the wall upon its foundation is sufficient to resist expansion, and of course this resistance increases the greater the distance from the free ends of the wall, reaching a maximum at the center.

This resistance to sliding is given by the formula

$$p = \frac{w\,(l\,h\,b)\,F}{144\,h\,b} = \frac{w\,l\,F}{144} = \frac{l}{2}, \qquad (3)$$

in which

p = resistance (in lbs. sq. in. of wall cross-section) to sliding at a point l feet from the nearest free end of the wall.
w = weight of a cu. ft. of wall = 144 lbs.
h = height of wall in ft.
b = average thickness of wall.
F = coefficient of sliding friction assumed at 0.5.

By comparing $p = \frac{l}{2}$ with f in the table, it will be seen that in order to have $p = f$ we must have the tabular value of $f = \frac{l}{2}$ or $l = 2\,f$; and for brick a wall would have to be 1,600 ft. long in order to have a stress of 400 lbs. per sq. in. developed in a vertical joint at its center; while for granite the wall would be 16,000 ft. long for a similar stress of 4,000 lbs. at its center. Walls of length sufficient to produce such maximum stresses are extremely rare, and would probably yield by lateral bowing or deflection if they did exist much in the manner that brick pavements arch up in the summer. A good old 1 to 3 Portland cement mortar has a compressive strength of 2,500 lbs. per sq. in., tested in cubes, and doubtless far more in thin slabs. From these two considerations, it is apparent that the vertical joints near the center of a cement masonry wall do not fail by crushing where the wall is free to slide on its base and opposed only by sliding friction. But while this is true of a wall taken as a whole, it is far from true of the individual courses; for, to illustrate by assuming an extreme case, suppose that the coping stones of a wall are thin and under the action of a hot sun they experience a sudden rise of 50° F. more than does the masonry below; then it is not sliding friction alone that holds the coping course, but the shearing strength of the mortar binding the coping to the wall below, which for 1 to 3 Portland mortar is about 100 lbs. per sq. in., whence

$$p^1 = 100 \times 12\,l, \text{ in which} \qquad (4)$$

p^1 = total resistance in lbs. per sq. in. to shearing at a point l feet from the free end of a wall.

From this equation it follows that a wall need not be 4 ft. long to develop a compressive stress in a vertical joint of the coping equal to 2,400 lbs. per sq. in. under the conditions assumed, which stress is certainly not far from sufficient to cause crushing of mortar where the coping is of the slightest thickness, and certainly sufficient where it has ordinary thickness; for it must be remembered that

$$P = n\,f, \text{ in which} \qquad (5)$$

P = compressive stress developed in a vertical coping joint 1 in. wide by n inches high, f having its tabular value,

while opposed to P is the shearing strength p^1 along a plane 1 in. wide by l feet long. Assuming that the coping stones are each 2 ft. long and 6 ins. thick (n = 6), we have for the first vertical joint from the end of the wall for a 50° rise in granite:

$$P = n\,\frac{f}{2} = 6 \times 2{,}000 = 12{,}000. \qquad (6)$$

Opposed to which we have:

$$p^1 = 1{,}200\,l = 2{,}400 \qquad (7)$$

Obviously, shearing is certain to have taken place long before such a rise of temperature can have occurred, and the coping stone will have been pushed out over the end of the wall, leaving a gaping vertical joint upon the subsequent fall of temperature. It only remains for repointing to be done in the next cool weather to secure a still further movement of the end stones upon the next rise of temperature, and it does not require very many such repointings to move the coping stones several inches and open up wide gaps at the ends of the coping with progressively narrower ones toward the center of the wall. That this does not always occur is due solely to the fact that many walls are laid in warm weather, so that when the temperature falls the shrinkage opens all vertical joints slightly, since the tensile strength of mortar is not comparable with its compressive strength, being only one-tenth as great, and cannot overcome the shearing strength of mortar in bed joints.

It follows from this discussion that masonry walls in northern climates should not be laid in courses, especially where such walls are long, of granite, and laid during cold weather; and that alternate coping stones especially should be let down into the stone beneath if it is desired to secure a wall in which gaping vertical joints will not be formed.

It follows, also, that monolithic concrete walls are far less subject to destruction from temperature changes than is ordinary masonry, especially coursed masonry; for in concrete the shearing stress is not concentrated at a few planes of comparatively weak mortar, but is uniformly distributed over the stone as well as the mortar.

We shall next apply the principles of mechanics to the frost theory; and in doing so we shall have to assume that the pores in stone are practically uniformly distributed. This we may do without sensible error, for stone of aqueous origin consisted originally of innumerable small particles of clay, sand or shell of minute size with voids uniformly distributed.

Water upon freezing expands about 10%, or its coefficient of linear expansion upon solidification is about 0.035. The force developed by freezing a cube of water completely enclosed in a vessel is given by the well-known formula:

$$f = \frac{E\,S}{L}, \text{ in which}$$

S = 0.035 L.
L = length of a side of the cube.
E = modulus of elasticity of ice, assumed at 335,000 lbs. per sq. in.

Whence $f = 0.035\,E = 11{,}825$ lbs. per sq. in. (8) which is certainly an enormous force. We must not lose sight of the condition that the expansion must be resisted in every direction in order to secure the development of such a force. Ice differs from other solids in that the slightest pressure long applied causes it to melt and flow like pitch.

Water enters a rock through an open crevice or pore; let freezing begin, and the ice will expand in the direction of least resistance; that is, toward the still open door through which the water entered. Assuming, however, that a plug of ice having formed stops the expansion of new ice forming below, what happens? Obviously, the ice plug offers resistance to expansion, and such resistance is measured by the adhesion of the ice to the sides of the channel; and since water transmits pressure equally in all directions, the following formula expresses the pressure so developed:

$$p_1 = \frac{c\,l\,t}{a}, \text{ in which} \qquad (9)$$

p_1 = pressure in lbs. per sq. in. upon the walls of the channel, developed by the formation of new ice below on ice plug.
l = perimeter of channel in inches.
a = area in sq. ins. of channel cross-section.
t = thickness of ice plug in inches.
c = adhesive force (lbs. per sq. in) of ice plug to walls of channel, which, if the ice melts under the pressure, may be as low as zero, but if the ice has not time to melt under the sudden pressure of new ice forming, may be as high as 40 lbs. per sq. in. (Trautwine).

For a cylindrical channel, whose diameter is D, we have $l = \pi\,D$, and $a = \frac{\pi\,D^2}{4}$, whence

$$p_1 = \frac{4\,c\,t}{D} \qquad (10)$$

But the force developed by hydrostatic pressure in each semi-circumference for each inch of length of the cylinder is

$$p_2 = p_1\,D \qquad (11)$$

whence

$$p_2 = 4\,c\,t \qquad (12)$$

Equation (12) therefore gives the pressure upon the walls of a cylinder due to the formation of new ice under an ice plug; and it likewise gives the pressure in a channel whose cross-section is a square. Giving c even its maximum value in equation (12), shows that the pressure in the pores of rock is slight compared with the resisting power of the material forming walls of the capillary channels; while, as above stated, there is every reason to believe that c will more likely have a value approaching zero, and especially so as the value of t, and consequently the pressure, increases. In any case, no matter what the cross-section of the channel, or what the value of c, at the surface of the rock t = o, whence the pres-

sure $p_s = o$. When t = 1-10-in., p $\leq$ 16 lbs. per sq. in.; when t = ½-in., $p_s \leq 80$ lbs. per sq. in., and so on, the pressure increasing gradually as the center of the rock is approached. Under such conditions, it is evidently impossible for water saturated rock to rupture upon freezing. Tests upon pieces of weak mortar, clay and soft brick saturated and frozen solid, confirm the mathematical deduction. Pumice, a volcanic scoria, one of the most porous of rocks, when exposed to freezing in volcanic regions, shows little signs of weathering. Shale, a much less porous but laminated stone, quickly crumbles under similar conditions. Why?

Referring to equation (9), we have for a rectangular channel that is very narrow as compared with its width, its cross-sectional dimensions being b and d,

$$l = 2d + 2b \quad (13)$$

$$a = b\,d, \quad (14)$$

which values substituted in equation (9) give

$$p_1 = \frac{c\,t\,(2d + 2b)}{b\,d} \quad (15)$$

If b is very great compared with d, as is the case in a rock seam or fissure, then d may be dropped in the numerator of equation (15) without sensible error, whence

$$p_1 = \frac{2\,c\,t\,b}{b\,d} = \frac{2\,c\,t}{d} \quad (16)$$

Assuming c = 40, we have

$$p_1 = \frac{80\,t}{d} \quad (17)$$

From equation (17) it is evident that the thicker the ice plug and the narrower the seam, the greater will be the unit pressure. If t = 10 ins. and d = 1-10-in., p_1 = 8,000 lbs. per sq. in., while if d = 1-100-in., p_1 = 80,000 lbs. per sq. in., provided c would remain equal to 40, which, of course, it would not under such pressure. However, equation (17) indicates the possibility of development of great disruptive pressure in a fissured rock, or in a laminated rock, once the layers have become slightly separated. The initial separation of laminations is undoubtedly due in greatest measure to alternate expansion and contraction of the rock under varying temperatures The solvent action of water may also aid in opening seams, but at best solvent action is exceedingly slow, except in limestone, or in cities where acid fumes exist.

We may conclude from the foregoing: (1) that mortar in vertical joints, especially near the ends of a wall and near its top, will fail soonest; (2) that broken range masonry will be more durable than coursed; (3) that concrete, being an extreme form of broken masonry, will be exceedingly durable if cement is good; (4) that face stones, if laminar, should be laid on their "quarry bed" to prevent heat stresses from shearing off the laminations, and not merely to better resist vertical pressure, as commonly stated; (5) that porosity in itself is no objection in a rock, but that very porous natural stones, being usually laminated, are less durable, while very porous artificial stones, like brick, are exceedingly durable; (6) that filling the pores of stone with paraffine or the like will not prolong the life; (7) that close jointed masonry is not more durable than wide jointed masonry; and, finally, that heat developed stresses, and not frost, are the primary cause of masonry disintegration in northern climates.

INTERMITTENT FILTRATION PLANT FOR THE SEWAGE OF RIDGEFIELD, CONN.

About the first of August, of the present year, the borough of Ridgefield, Conn., put in operation a sewage disposal plant of the intermittent filtration type. The plant is worthy of note, because comparatively few places of so small a size have sewerage systems, much less works for purifying their sewage. The following description of the disposal works is drawn from the report of the Connecticut State Board of Health, and from a letter to this journal dated September 3, 1902, from Mr. T. H. McKenzie, M. Am. Soc. C. E., of Southington, Conn., who was the engineer for the sewerage and sewage disposal systems.

Ridgefield lies at an average elevation of 850 ft. above tidewater, and is becoming popular as a summer resort. Its permanent population is put at 2,500, with 800 additional in the summer. A public water supply was introduced by a private company in September, 1900, and $50,000 of bonds for sewerage purposes were authorized by the State legislature early in 1901. In May, 1901, a borough organization was completed. In the following October sewer construction was begun, and about August 1, 1902, sewage was admitted to the filter beds.

The plans called for about eight miles of 15 to 8-in. sewers. The soil encountered in laying the sewers was a heavy clay and hardpan, underlaid by quicksand. Ledges and many boulders were also found.

The sewer pipe was in 3-ft. lengths, with deep sockets, and with oakum and cement joints. No surface or roof water will be admitted to the sewers. For flushing the sewers, iron pipes are laid from each 12-in. line to a brook, where gates are placed. The laterals will be flushed from fire hydrants or from water carts.

The filtration area chosen was a gravelly knoll of about four acres, some 3,000 ft. from the railroad station and 1,600 ft. from the nearest houses. On three sides of the knoll are woods, but immediately about it, and 9 ft. below it, is a swamp.

Of the four acres of filter beds, three acres have been provided with 5 and 6-in. underdrains, laid 4 ft. below the surface and 3 ft. above the swamp level. The drains are 25 ft. apart, but extra lines, for which pipe is on the ground, will be laid some time in the future, making the drains 12½ ft. c. to c.

There are seven beds of unequal size and varying shape. The 15-in. outlet sewer terminates in a distributing well at the corner of four beds, and there are other distributing wells, with connecting pipes in the embankments, all so located as to serve the whole area. Outlet pipes 8 ins. in diameter, controlled by shear gates, extend from the wells to the beds, which latter slope slightly from the outlets to facilitate the distribution of sewage over the surface.

The land purchased for the filter beds has an actual area of six acres. Deducting embankments, 3½ acres are available and have all been laid out in filter beds. Including engineering and superintendence, the filter beds have cost a total of $6,000. This includes the pipe bought for the additional underdrains and not yet laid. The bonds issued to pay for the sewers and disposal area run for 20 years, and bear 4% interest. The entire cost of the sewers and filtration works, including engineering, is $54,000. The six acres of land cost only $150. Mr. Geo. E. Benedict is borough clerk of Ridgefield.

BOOK REVIEWS.

ARITHMETIC OF ELECTRICAL MEASUREMENTS, With Numerous Examples Fully Worked.—By W. R. P. Hobbs, R. N., Associate of the Institute of Electrical Engineers of Great Britain; Revised to Date by Dr. Richard Wormell, M. A. Ninth Edition. London: Thomas Murby; New York: D. Van Nostrand Co. Cloth; 4½ × 7 ins.; pp. 112. Price, 50 cts.

This little book is devoted to arithmetical problems in connection with electrical work and their solution. The problems are elementary and the first three-fourths of the book relate solely to battery currents. In the concluding chapters some simple problems relating to direct current as used for electric lighting and electric motors are given.

MALARIAL FEVER.—Its Cause, Prevention and Treatment. Containing Full Details for the Use of Travelers, Sportsmen, Soldiers and Residents in Malarial Places. By Ronald Ross, Lecturer in the Liverpool School of Tropical Medicine. Ninth Edition, Revised and Enlarged. New York: Longmans, Green & Co. Cloth; 6 × 9 ins.; pp. 68; illustrated. 75 cts. net.

This little volume, by an eminent authority and investigator, has been revised so as to include the most recent available information on the subject, including the relation of mosquitoes to malaria. It should be of use to municipal health officers and also to engineers, campers and others working or living in malarious districts.

HEATING AND VENTILATING BUILDINGS.—A manual for Heating Engineers and Architects. By Rolla C. Carpenter, M. S., C. E., M. M. E., Prof. Experimental Engineering Cornell University; 4th edition, revised and enlarged. New York; John Wiley & Sons, 1902; 6 × 9 ins.; cloth; pp. 502; 277 figures; $4.

The first edition of this work was published in 1895, and the present edition has about one-third more pages than the first. Three new chapters have been added, one on the fan or blower, a second on the general subject of mechanical systems of heating and ventilating, and a third on school house heating and ventilation. The first edition of this work was reviewed in our issue of July 9, 1896, and certain criticisms made then seem still to hold good. Of these perhaps the most important is that on p. 52 of the new edition, where the coefficient of linear expansion of air is given at $\frac{1}{460}$ for each degree Fahrenheit, where, as a matter of fact, it is $\frac{1}{491}$. We observe also that the reference numerals of illustrations from p. 411 to p. 428 do not correspond with those given in the text, causing not a little confusion to the reader.

The chapter on electric heating is particularly interesting in bringing out the fact that the coal consumed in generating electricity for such heating is about 15 times as much as the coal consumed in supplying an equal amount of heat with furnaces.

The work as a whole forms an excellent manual for the use of architects and engineers interested in heating; but inasmuch as it is primarily written for men of scientific training, it would better serve their purposes if it were written in more concise style. As an illustration of the class of paragraphs that are all too common in this, and in many other scientific works, we quote as follows from the chapter on School House Warming:

> A rule which may be safely insisted upon for general adoption and application is that pure air should be supplied to enclosures in the maximum rather than in the minimum quantity tolerable. Only two considerations should be allowed to limit the quantity of air supply; air-draughts and bank-drafts.
>
> Draughtness in air currents is far more dangerous to health than the ordinary vitiation of air in badly ventilated enclosures. On the other hand, the warming and, under some circumstances, the moving of air in large quantities for ventilating work is far from costless, etc.

There is not a fact in this quotation not known to any one of ordinary education and common sense. The insertion of such matter in scientific works serves only to add to their cost and bulkiness.

THE MINERAL INDUSTRY.—Its Statistics, Technology and Trade in the United States and Other Countries to the End of 1901. Founded by the late Richard P. Rothwell; now edited by Joseph Struthers, Ph. D., Columbia University, New York. Vol. X. New York and London: The Engineering and Mining Journal. Cloth; 6¼ × 9¼ ins.; pp. 982. $5.

The tenth yearly issue of this statistical history of the mineral production of the world records the unprecedented production of minerals in the United States during the year 1901 as having been valued at $1,193,068,461, which is some thirty-seven million dollars greater than the preceding year.

As in previous issues, each mineral is taken up alphabetically, and all the available data relating thereto are given.

Besides the statistical information, there are about 70 articles written by experts on the various subjects treated.

The history of the development of asphalt for paving is given in great detail. An interesting use of the copper tailings of a mill at Lake Superior is for the making of asphalt pavement, the proportions of which were 70% sand tailings, 15% limestone and 15% asphaltum cement. One company used 15,000 tons of tailings in this way in 1900, and 20,000 tons in 1901.

An article on "The Slag Cement Industry in the United States," by E. C. Eckel, contains much valuable information. Civil engineers are inclined to be prejudiced against slag cement, due to the fact that it has often masqueraded under the name of "Portland," or has been adulterated with unburned slag. Mr. Eckel says that slag cement is far superior to Rosendale cement in strength, and but little inferior to Portland, and in price averages about 25% higher than Rosendale. It is also stated that slag cement does not stain masonry, which, if so, makes it especially desirable for architectural purposes. Slag cements are notably deficient in the property of resistance to wear, as in sidewalks, but are well adapted for use in pavement or other foundations. There are seven slag cement factories in the United States. A 30-page paper on the "Manufacture of Coke (and By-products) in the United States," by F. Schniewind, covers the subject thoroughly in brief space.

A chapter reviewing the literature of ore dressing in 1901 is of especial value to mining engineers and mill superintendents.

A very full and well-arranged index makes easy the task of finding information upon any mineral.

THE BALANCING OF ENGINES.—By W. E. Dalby, M. A., B. Sc., M. Inst. C. E., Professor of Engineering and Applied Mathematics in the City and Guilds of London Institute. New York: Longmans, Green & Co. Cloth; 8vo.; pp. 283; 173 illustrations. $3.75 net.

The subject of balancing steam engines has recently become one of great importance, especially to marine and locomotive engineers, on account of the vibration and other difficulties caused by the high piston and rotative speeds of modern practice. The literature of the subject is scattered through the transactions of various societies hard to find and difficult to read. Prof. Dalby has therefore done a good service to the profession in putting in

book form the excellent treatise which is before us. In his preface the author says:

The main object of the book is to develop a semi-graphical method which may consistently be used to attack problems connected with the balancing of the inertia forces arising from the relative motion of the parts of an engine or machine. In the case of a system of revolving masses, or a system of reciprocating masses where the motion may be assumed simple harmonic without serious error, the application of the method is simple in the extreme, as it requires nothing but a knowledge of the four rules of arithmetic and good draftsmanship. Moreover, the work can be easily checked, and in the case of symmetrically arranged engines, like locomotives, for example, the method is self-checking.

The book is not easy reading, because the subject is not an easy one. To master it, one should be well versed in mathematics, including calculus, and in applied mechanics, and he should make himself familiar with vector quantities by a careful study of the first chapter of the book, in which they are briefly but clearly taught. With this equipment, the student will find the subject no more difficult than Rankine's "Applied Mechanics," for instance, of which book the author's style reminds us, although it is not so concise, and therefore more readable, than Rankine's. The headings of the principal chapters give an idea of the subjects covered. They are: The Balancing of Revolving Masses; The Balancing of Reciprocating Masses—Long Connecting Rods; The Balancing of Locomotives; Secondary Balancing; Estimation of the Primary and Secondary Unbalanced Forces and Couples; The Vibration of the Supports; The Motion of the Connecting Rod.

The book concludes with a set of 57 exercises for students, the answers being given. We give the following as an example of the kind of problem that may be solved by the method given in the book:

The cylinders of a four-crank engine are arranged symmetrically. The pitch of the outer pair is 39 ft. and of the inner pair 15 ft. The mass of each set of reciprocating parts belonging to the outer cylinders is 6 tons. Find the crank angles and the inner masses, so that the reciprocating masses may be in balance for primary and secondary forces, and primary couples.

We regret to see that the author makes use of the objectionable term "poundal," which has been so well criticised by Prof. Perry in his "Calculus for Engineers," and also the term "lbs. weight," instead of "lbs.," for the value of a force. We quote:

If M is the mass in pounds of the moving body, and A the acceleration produced by the force F, acting at the mass center,

$$F = M A \text{ poundals, or } F = \frac{M A}{g} \text{ lbs. weight.}$$

Since in balancing problems the magnitudes of the forces are generally not concerned, it is more convenient to use the first expression, avoiding thereby the introduction of g into the work.

It would have been just as easy to get rid of g and at the same time obtain the magnitude of the force in pounds by the good old English method, viz.:

F (in lbs.) = M A, in which M = W ÷ g, W being the weight of the body in pounds as determined by an even-balanced scale anywhere on earth, and g the acceleration due to gravity at London = 32.2.

PROGRESS ON THE ST. LOUIS EXPOSITION.

Our readers will doubtless be surprised to learn how far toward completion the grading and building has been pushed on the Louisiana Purchase Exposition grounds at St. Louis, although the exposition will not be opened until 1904.

The area included in the grounds is about 1,200 acres; and the earth to be moved for lakes, ponds, roads, building sites, etc., amounts to 1,500,000 cu yds., of which 80% has already been excavated. Almost 9 miles of water pipe for domestic service, and nearly 8 miles of sanitary sewers have been laid, while more than 3 miles of fire service pipes are being installed.

There are nearly 4 miles of macadam and telford roadway, 30 ft. wide, nearing completion, and some 9 miles of railways are under construction on the grounds.

Of the nine buildings forming the nucleus of the Fair, for which $5,000,000 has been appropriated, there are seven in course of construction and but one for which the contract has not yet been let. The Textiles Building and the Electricity Building will both be completed by the end of this year. Bids for the extensive U. S. Government buildings will be opened Nov. 10. In addition to the regular Fair buildings, eleven new buildings have been leased from Washington University, seven of which are finished, and being impressive in appearance serve admirably for the uses to which they will be put.

Sites for 26 State and Territory buildings, and for four buildings for societies have been allotted. These sites are on the high and commanding ground in the southeast corner of the exposition grounds.

The demand from foreign governments necessitates an immediate assignment for each.

Considerable electrical work has been done; the main temporary buildings have been wired; power lines have been established, and power service is being furnished to those contractors requiring it. Almost ten miles of 3-in. conduit for wire distribution has been placed in the ground. A subway, 7 or 8 ft. in size, is practically completed, and extends through the heavy service portions. A great amount of material for electrical installation has been delivered at the grounds. The area that will be under the roofs of the 14 principal buildings, exclusive of courts and galleries, is 117 acres.

THE NEW YORK RAPID TRANSIT RAILWAY.

XII.

Contract Section V-b.

Contract Section V-b of the Rapid Transit railway extends from the center line of 47th St. and Broadway north along Broadway, and across the Columbus Circle to the center line of 60th St. The subcontractors are Naughton & Co., of New York city, who began work Sept. 20, 1900.

MATERIALS PENETRATED.—The subsoil underlying Broadway, between 47th and 60th streets, is rock, approaching close to the street surface for much of the distance, but having overlying

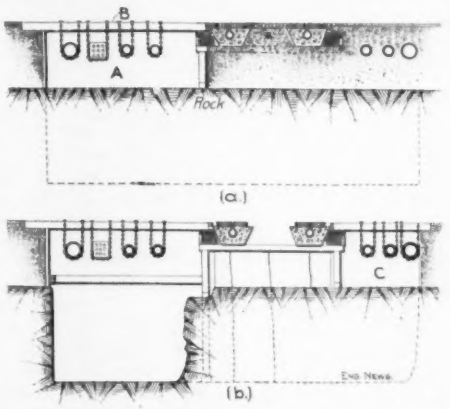


Fig. 62. Diagram Showing Method of Excavation of Section V-b.

pockets of earth in places. Such a pocket of earth was encountered for about 400 ft. north of 47th St., and another at about the center of the Circle, under and adjacent to the Columbus Monument. Generally speaking, the section can be defined as a rock section, the rock being a mica-schist of considerable hardness and solidity.

STREET AND ADJACENT STRUCTURES.—The surface of Broadway is occupied by a double-track conduit electric railway for the whole length of the section. Beneath the street surface there were the customary water and gas pipes, sewers and other conduits. Above the surface there were only two places on the section at which the structures interfered with straightforward methods of work. These were at 53d St., where two columns of the Manhattan Elevated Ry. crossing came directly over the subway, and at the circle, where the tall shaft monument to Christopher Columbus came partly over the subway roof. The buildings along the route were of such a character and so situated as to cause no difficulties.

SUBWAY STRUCTURE.—The subway structure throughout Section V-b is the standard four-track construction. In plan it follows the center line of Broadway closely, and therefore has quite a proportion of curved track, but the curves are all very flat. In profile the grade is slightly ascending the whole distance. There is one station located just north of the center of the Columbus Circle and one at 50th St.

METHODS OF WORK.—The method of work adopted was as follows: A trench was first opened between the curb and the nearest surface track and sunk to subgrade. This trench was next widened by excavating laterally part way under the surface tracks, they being supported in the meantime by needles and posts. In this excavation one-half of the subway structure was erected and the back-filling was completed. A second trench was then opened on the opposite side of the street and widened laterally in the same manner as was the first trench. In this excavation the second half of the subway structure was erected and the backfilling completed. The details of the various operations were as follows:

Referring to Fig. 62, the first operation was to open the trench A to rock between the nearest surface track and the curb. As soon as the street paving had been removed in cutting this trench the cross braces, B, were set in place and notched into the street surface, as indicated in Fig. 62a, and as shown clearly by the view, Fig. 63. To these cross-braces the subsurface conduits were hung by means of slings. After the excavation of trench A had been completed to rock, drifts were extended laterally from its inner edge under both tracks of the street railway. These drifts were spaced about 10 ft. apart, and in them were inserted 12 × 12-in. × 25-ft. needle timbers. The near ends of these needles were carried on vertical struts resting on the rock surface. The trench was then carried down through the rock to subgrade, as indicated by Fig. 62b. In taking out this rock the working faces were carried diagonally across the trench, the work being done by drilling and blasting. The succeeding step was to widen the trench laterally under the tracks by making the cuts indicated by the dotted lines in Fig. 62b. As these cuts were made, the needle beams were supported by introducing posts under them. When a sufficient length of trench had been opened and widened, as described, two bays of the subway were erected in the excavation, masonry piers were built 10 ft. apart on the roof to carry one surface track, the space between the piers under the railway track being filled with stone laid in by hand, and the backfilling was completed.

The second stage of the work was commenced by opening the trench, C, Fig. 62b, to rock and placing shores under the ends of the needle beams and braces across its top. This trench was in turn carried down through the rock to subgrade and widened laterally by operations similar to those carried out in the first trench and described above. The second half of the subway structure was then erected and connected to the first half, rubble masonry piers were built on its roof, as described above, to carry the second line of street railway track, and the needles were removed and backfilling completed.

Figs. 63 to 66 are views of the work in progress. Fig. 64 shows the first trench opened to subgrade, the struts supporting the ends of the needle beams, the method of slinging the street pipes from the trench timbers, and work in progress laying the concrete floor of the first two bays of the subway. Fig. 65 shows work in progress trimming down the side of the trench nearest the building line to prepare for the construction of the subway side wall, portions of which the view also shows in place. The nature of some of the street conduits encountered and the manner of supporting them during construction are again shown by this view. Fig. 66 is a view showing the first bay of the steelwork in process of erection.

SUPPORT OF ELEVATED RAILWAY.—At 53d St., the route of the subway is crossed by the double-track structure of the Manhattan Elevated Ry., two of its columns coming directly over the center row of subway columns, and between the two tracks of the street surface railway. To build the subway structure means had of course to be provided for the temporary support of the elevated structure during construction, and for restoring the original foundations after construction. The method adopted was to support and restore each column separately, but in the same manner.

Considering only the south column, in order to avoid confusion, the mode of procedure was as follows: Two trenches were excavated transversely across Broadway and the subway structure, about 35 ft. apart; one trench was located about midway between the two columns and the other was located about 17 ft. south of the south col-

FIG. 63. VIEW SHOWING FIRST OPERATION IN EXCAVATING FOR SECTION V-b.

umn. Each trench was made 6 ft. wide and extended under the Broadway surface tracks and down to subgrade. When excavation was completed the concrete floor of the subway was laid on their bottoms. A trestle-bent was next erected in each trench, having the form and dimensions shown by Fig. 67a. Meanwhile a trench along the center line of the street between the surface railway tracks was excavated down to the depth of the tops of the trestle bents. Two 30-in. plate girders, about 35 ft. long, previously used to support the Columbus Monument, were lowered into this trench and skidded carefully into the positions A and B, shown by Fig. 67a. The placing of these girders was facilitated by the fact that when the elevated column foundation, shown by dash lines in Fig. 67, had been built an opening was left through it to allow for the passage of a 12-in. and a 20-in. water main. These mains had subsequently been removed, leaving the opening free, and advantage was taken of this to insert girder B, with but a slight cutting away of the foundation masonry. Girder A was so located that it encroached only slightly on the foundation masonry and made only a small cutting away necessary. Fig. 67b shows the girders in elevation.

After the girders had been placed a grillage of five 15-in. I-beams was placed across them and bolted together with separators. A platform of 12 × 12-in. yellow pine was then placed on the top of these beams and uprights and batter posts placed on this platform and extended up to about 4 ins. below the underside of the cross girders of the railway. Braces of 3 × 10-in. yellow pine were then bolted through the caps and sills of the uprights and batter posts, and also through the posts themselves, thus tieing the whole timber structure together. The girders were tied together on each side of the grillage by 4 × 5-in. angles, one being riveted to the top cover plates and one to the bottom plates with diagonals of 4 × 5-in. angles riveted to each. These operations practically completed the supporting structure, as shown by Fig. 67.

The next task was to complete the subway excavation and subway structure, and to restore the column foundations. The nuts on the anchor bolts through the cast-iron base were first loosened and the column and cast-iron base were then raised off of the brick pier 1 in. by jacking-up under the cross girders, and the load was then transferred to the wooden uprights by wedging between cross girder and uprights. The structure was left to stand for 24 hours with traffic going over it. The position of the cross girder at the end of that time being about ½-in. above its original position. The excavation around the pier and the tearing out of the same was then commenced and carried down to subgrade. As the excavation proceeded in depth the trestle bents were braced north and south as occasion required.

After the excavation was completed, the concrete floor and waterproofing were put in place and the iron erected. The three subway columns which carry the pier supporting the elevated column were spaced 4 ft. 6 ins. on c. to c., and the roof beams were increased from 18 ins. to 20 ins. in depth. The arches of the roof were then put in place and waterproofed, and the roof was ready to receive the pier. Wooden forms were put up between the girders to make the pier, and anchor bolts with plate washers on their bottoms were hung from the base, the bottoms of the rods being about 6 ins. above the waterproofing. Concrete was then put into the form to the level of the plate washers, and three 15-in. I-beams, 9 ft. 6 ins. long, were placed in the concrete, as shown by Fig. 68, the outside beams resting on the plates. The forms were then filled with concrete to about 2 ins. below the column base and allowed to set for four days. The space between the base and concrete pier was then filled with stiff grout, and the column loosened from the cross girders, and together with the base, lowered about ½-in. into the grout to its original grade. After eight days the cross girder was lowered onto the column and the timber uprights removed.

The girders were then moved forward to a similar position under the north column, where the work proceeded in a similar manner. The dead load of the elevated structure is given as 40,000 lbs., and the live load as 100,000 lbs., while the load on each of the three columns under the pier is about 120,000 lbs.

SUPPORT OF COLUMBUS MONUMENT.—As already stated, the subway crossed the circular plaza at the southwest entrance to Central Park and passed under one edge of the foundation for the Columbus Monument, located at the center of the circle. This arrangement consists of a shaft carrying a statue of Columbus and mounted on an elaborately ornamented pedestal from which three steps lead down to the street level and forms a base. The base of the monument is a 35-ft. square with the corners cut off, and it rests on a foundation 45 ft. square and 14 ft. deep. The foundation consists of 12 ft. of rubble masonry resting on 2 ft. of concrete. From the top of the rubble masonry to the top of the shaft is 75 ft. The weight of the whole monument, excluding foundations, was estimated to be 724 tons. The subway as planned cut under the column base some distance and penetrated the foundation masonry. Fig. 68 shows the relative location of the column foundations and the subway structure. The problem before the contractors was, of course, to build the subway without damage to the monument, and the manner of solving it is indicated by Figs. 69, 70 and 71.

The material on which that portion of the monument over the subway rested, was gravel mixed with sand, which stood fairly well when kept dry. It was expected that rock would be encountered under the foundation, as it had been found just north and also west of the monument, at about 3 ft. from the surface of the street, and on the

FIG. 64. VIEW SHOWING TIMBERING FOR SURFACE RAILWAY TRACKS, SECTION V-b.

south side about 8 ft. from surface, but as none was found it is probable that the rock dips from the westerly side towards the east at about 45°, a fact which would not increase the stability of the monument when the easterly half was undermined.

Shafts were sunk on the south and north sides of the foundation of the monument, and just west of the west line of the subway, to a depth of about 3 ft. below the grade of the bottom of the subway. From these shafts a tunnel 6 ft. wide and 7 ft. high was driven under the foundation of the monument, the east side of this tunnel corresponding with the west side of the subway, and the roof of the tunnel being the bottom of the foundation of the monument. After the tunnel had been driven through it was filled with concrete to the subgrade of the subway, and uprights of 12 × 12-in. yellow pine were placed about 5 ft. apart along the tunnel and firmly wedged from the bottom of the monument foundation with oak wedges. Commencing at the center of the tunnel and working both ways, it was then filled with rubble masonry, care being taken to build the masonry solid to the bottom of the foundation

FIG. 65. VIEW SHOWING TRIMMING OF SIDE OF EXCAVATION AND CONCRETE SIDE WALL CONSTRUCTION IN SECTION V-b.

of the monument. This completed the underpinning of the monument.

While the above has been going on the excavation for the tunnel subway had been carried on along the north, east and south sides of the monument to within 3 ft. of its sides. Commencing at the east side of the monument and at a height of about 4 ft. above the finished roof of the subway and working towards the west, a channel about 6 ft. in depth was cut out of the foundation until the monument overhung about 5 ft. Four uprights of 12 × 12-in. timber were then placed in this channel under the outer edge of the monument, and the cutting of the channel then continued until it reached the westerly line of the subway. In the meantime the permanent concrete floor had been placed on the north and south sides of the monument, and trestle bents with 12 × 12-in. caps, uprights, and batter posts erected to carry the girders. These girders were the same that were used to support the elevated railway at 53d St., and were skidded into position from the south side of the monument, and through the channel cut in the foundation. Over the top cover plate of each girder a 1-in. plank was placed to equalize the unevenness due to the rivets, and any holes between the girders and the monument were filled with fish-plates to give an even bearing. A 60-ton hydraulic jack was then placed alternately under the end of each girder, and iron plates were then driven in between cap and trestle and bottom of girders.

The uprights in the outside of the channel were then knocked out and the remainder of the foundation and the earth below it were excavated to the grade of the subway. The west side of the foundations and channel as cut out, together with the side of the rubble underpinning, were then plastered with cement mortar to make a smooth surface to receive the waterproofing which envelopes the subway. The steel columns and beams for the subway were then placed in position and the arches in the roof and sidewall (which at this point the south end of the Columbus Circle station are of enameled brick backed with concrete) were laid and the construction of the subway completed around the monument, except where the trestle bents came through a panel of the subway roof.

Along under the east side of the monument foundation, and on the finished roof of the subway, a rubble masonry pier about 4 ft. thick was then built, a course of "wedging" stone being put in about 3 ft. above the roof of the subway; between these "wedging stones" iron wedges were driven, thus transferring a portion of the load on the girders to the roof of the subway. The west girder was then taken out and rubble masonry built in (and wedged as above) between the roof of subway and base of monument. The remaining girder was then removed, another pier built under the foundation, where it had been, and masonry was then also built along the north and south sides from the old foundations to the end of the piers just mentioned, thus enclosing the monument and leaving it supported on the roof of the subway.

PLANT.—Compressed air is used for power throughout Section V-b. The power plant was located on a vacant lot at the south side of the circle, where a small storage yard, the contractors' field office and some minor buildings were also placed. Fig. 72 is a diagram plan of the power plant. As will be seen, it consists of a frame building divided into an engine and a boiler room. The boiler room contains two 125-HP. and two 100-HP. boilers of the return tubular type. These boilers supply steam to three Ingersoll-Sergeant air compressors, which pump to a 54-in. × 12-ft. receiver. From the receiver an 8-in. pipe extends to the street and there bifurcate into line pipes running north and south. The line pipe going north is about 600 ft. long and 4 ins. in diameter and that going south is 5 ins. for 1,500 ft. and 4 ins. for 1,500 ft. Expansion joints are placed every 1,000 ft., and drip pockets at all low points. Connections are inserted in the line pipes every 50 ft.

The field plant consisted of four Carson-Lidgerwood cableways, and ten stiff-leg derricks for hoisting the material, 36 drills, 4 pumps, and of pneumatic tools for erecting the steelwork. All of these machines are operated by compressed air.

The work of Section V-b was carried out under the direction of Mr. Alfred Craven, M. Am. Soc. C. E., Division Engineer, Rapid Transit Commission, and Mr. C. W. S. Wilson, Assoc. M. Am. Soc. C. E., Engineer for the contractors.

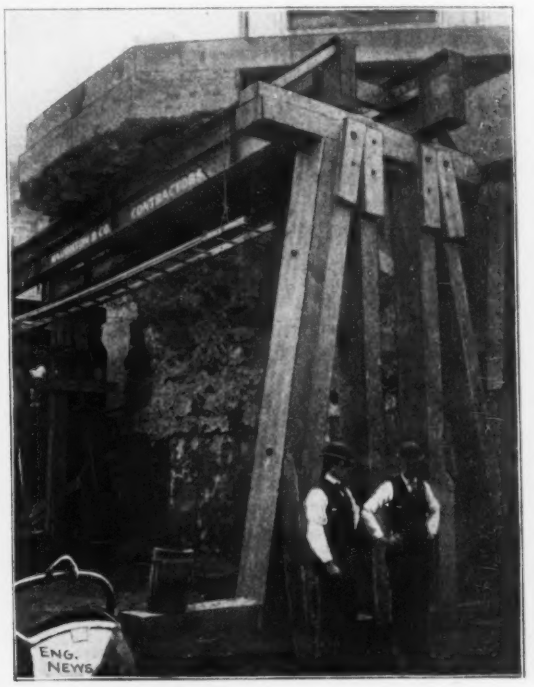


Fig. 71. View Showing Steel Girder and Trestle Bent Support for Columbus Monument.

A STUDY OF ROAD-BUILDING GRAVELS.

By Ira O. Baker,* M. Am. Soc. C. E.

Gravel roads are quite common, and although not equal to first-class broken stone, they serve a most excellent purpose; but apparently there is in engineering literature no statement of the conditions fitting gravel for road building purposes. Probably this is in part due to the fact that gravel roads are usually built of the most convenient local material without much consideration of the principles involved. There is, however, often a choice of gravels, and with a view of determining the basis for an intelligent selection of the material, the writer has recently made a study of a number of well-known road building gravels. Before giving the immediate results of this study, he will state what he conceives to be the fundamental requisites for a good road gravel.

REQUISITES FOR ROAD GRAVEL.

To be suitable for road building purposes, gravel should fulfil the following conditions: (1) The fragments should be so hard and tough as not easily to be ground into dust by the impact of the wheels and hoofs; (2) the pebbles should be of different sizes, each in the proper proportion; and (3), there should be intermixed with the coarser particles some material which will cement and bind the whole into a solid mass.

1. DURABILITY.—Having been transported considerable distance by water and ice, gravel is usually fairly durable, since the softer and more friable fragments have been worn away. Although gravel is not equal to the best crushed stone for road building, in many parts of the country the rocky fragments transported by the water and ice are more durable than any of the native rocks. From the nature of their origin, it is apparent

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that gravel may differ widely in the character of the stones composing it. Not only do different gravels differ from each other, but any particular gravel may be composed of fragments of a variety of rocks. Since the gravels of any locality are likely to contain substantially the same rocky fragments, there is not much opportunity for choice

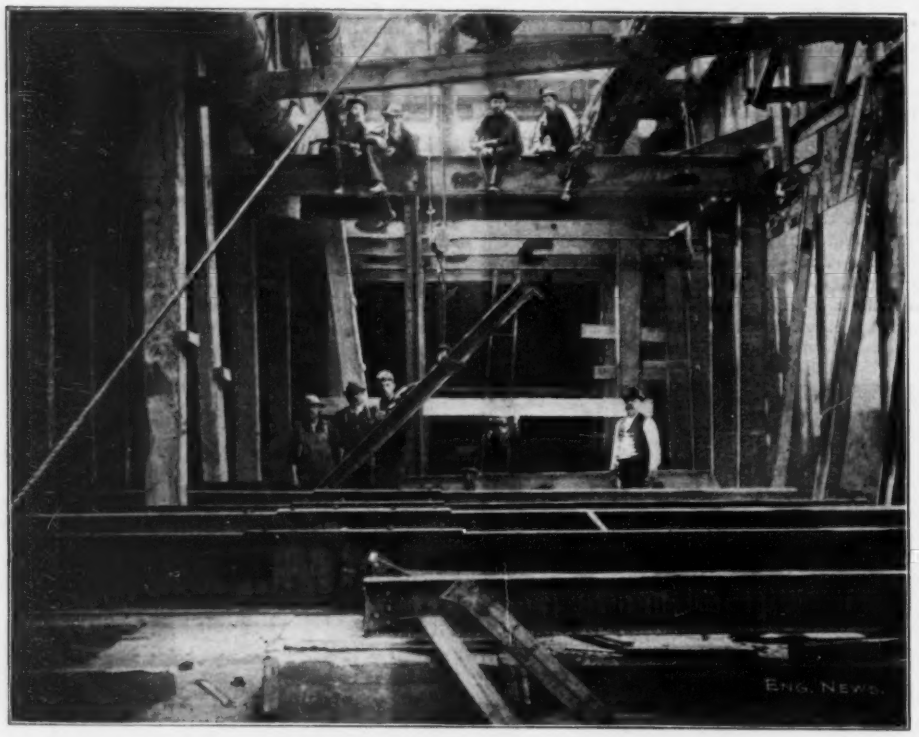


FIG. 66. VIEW SHOWING ERECTION OF STEEL WORK IN PROGRESS ON SECTION V-b.

under this head; and therefore this phase will not be treated here, although it will be considered incidentally later.

2. SIZES.—If the fragments are too large, the road will not be homogeneous, and the large stones will work to the surface under the action of the traffic and frost; but, on the other hand, if the pebbles are too small, the gravel will partake too much the character of sand and will be difficult to bind properly. The best results are obtained when the largest pebbles are not more than ¾ in. to 1 in., or at most 1½ ins., in greatest dimension. With stones larger than 1 in. it is difficult to keep the surface from breaking up when dry. Small gravel makes a pleasanter road and one that is easier to keep in order. If stones larger than 1½ or 2 ins. are present, they may be screened out and be used in the foundation.

It is desirable that the several sizes should be so proportioned that the smaller ones are just sufficient to fill the interstices between the larger ones, since then less binder is required. The binder is usually the least durable ingredient, and hence the less there is of it the better. Gravel can often be improved by screening—either to remove an undesirable size or to separate it into several sizes afterward to be combined in new proportions. The proper proportion depends upon the nature of the gravel—whether the binding material is already present in the form of dust, or whether some of the pebbles must be crushed to produce the binder.

3. BINDER.—The most important requisite for good road-building gravel is that it shall bind or pack well. If it does not pack well, the wheels will sink into the gravel and increase the force of traction, and the rain will penetrate the road bed and soften it. To bind well, the several fragments should be in contact with one another at as many points as possible, in order that they may be firmly supported, and that friction may act to the best advantage to resist displacement. To secure contact at every point, all the interstices between the fragments should be filled—those between the large pebbles, with small pebbles; those between the small pebbles, with sand grains; and, finally, those between the sand grains, with some finer material, called a binder. The binding material must be very finely divided, so that it can be worked into the smallest interstices, and for this reason it is the least durable part of the gravel, being easily washed out or blown away. For the best results, then, the sizes of the coarser particles should be so adjusted as to require a minimum amount of binder.

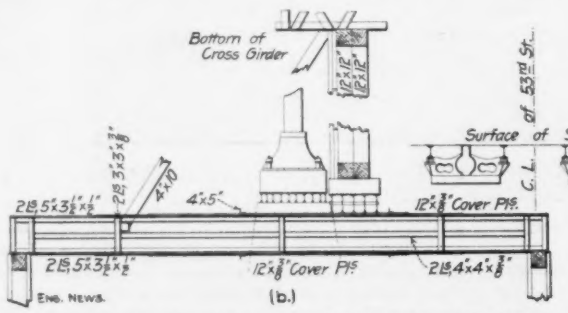


FIG. 67. METHOD OF SUPPORTING ELEVATED RAILWAY AT 53D ST., SECTION V-b.

TABLE I.—Sieve Analyses of Road-Building Gravels.

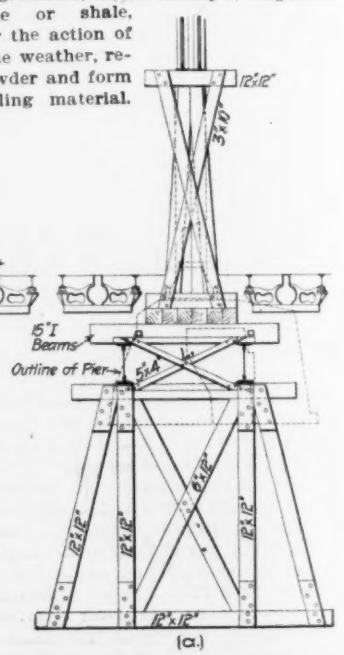
Ref. No.	Size of mesh.	1. Urbana.	2 Decatur.	3. Lexington.	4. Rockford.	5. Peekskill.	6. Buck Hill.	7. Rock Hill.	8. Shark River.
		Per cent.							
1.	Caught on 2-in. mesh	0.0	0.0	0.0	6.0	0.0	0.0	0.0	0.0
2.	" " 1-in. "	0.3	1.1	4.6	35.6	2.1	0.0	0.0	0.0
3.	" " ½-in. "	9.6	12.2	10.9	23.5	41.3	11.7	3.3	0.8
4.	" " ¼-in. "	13.0	10.5	8.7	7.4	25.5	11.0	13.4	12.7
5.	" " 1/10-in. "	41.1	14.8	20.2	9.1	17.8	8.4	25.1	33.2
6.	" " 1/20-in. "	12.1	3.9	15.3	3.2	2.2	20.4	14.0	7.4
7.	" " 1/40-in. "	3.9	7.8	15.9	2.7	1.8	8.2	7.2	5.5
8.	Passing 1/40-in. "	16.2	40.0	21.0	8.7	8.4	20.2	16.3	20.0
9.	In supension	3.8	9.6	4.2	8.2	0.9	20.0	20.7	20.2
	Total	100.0	100.0	99.9	99.4	100.0	99.9	100.0	99.6
	Voids in washed gravel	25.5	30.5	27.3	29.6	30.5	28.3	34.0	28.2

The binding material may consist of clay, loam, silica, stone dust, iron oxide, etc., or some ingredient which will crush under traffic and furnish a fine dust. Clay is by far the most common material; but the only recommendation for it is (1) that it is easily reduced to an impalpable powder by the action of the wheels or by water, and (2) that it is often found already mixed with the gravel, and (3) that if it must be artificially mixed, it is plentiful and cheap. Clay is an undesirable binder, since its binding action depends in a large measure upon the state of the weather. During the rainy period it absorbs water and loses its binding power, and the road becomes soft and muddy; while in dry weather it contracts and cracks, thus releasing the pebbles and giving a loose surface. Clay is also very susceptible to the action of frost; and consequently when the frost is going out a gravel road with a clay binder ruts up badly and frequently breaks entirely through. When the weather is neither too damp nor too dry, a gravel road with a clay binder is very satisfactory. The clay should be no more than enough to fill the voids in the pebbles and sand, and for a good road gravel should not exceed 15 to 20% of the mass. Not infrequently much greater quantities are present. The surplus may sometimes be removed by screening; but often it can be removed only by washing—a process which is usually so expensive as to be prohibitive.

Loam is chiefly clay mixed with sand and a little vegetable matter, lime, etc., and as a binding material it has all the characteristics of clay. A very finely divided silica, easily mistaken for clay, is occasionally present in gravel, and makes an excellent binding material.

Iron oxide is frequently found as a coating on the pebbles in such quantities as to cement them firmly together. These ferruginous gravels, when broken up and put upon a road, will again unite—often more firmly than originally, because of the greater pressure—and form a smooth, hard surface, impervious to water.

Comparatively coarse gravel frequently contains some ingredients, as, for example, fragments of limestone or shale, which, under the action of traffic and the weather, reduce to a powder and form a good binding material.



Sometimes gravel contains bits of ironstone (clay cemented with iron oxide) in the form of thin flat chips which break and crush easily under the wheels, which, if present in any quantity, make a most excellent binding material.

The binding action referred to above is wholly mechanical, but there is another action which sometimes plays an important part in binding a gravel road. Laboratory experiments prove that if a fine powder of certain stones is wetted with water and subjected to compression, the particles adhere or stick together with considerable force. This action has not received much attention from road builders, having been studied only by the Massachusetts Highway Commission with refer-

once to broken stone roads; but this cementation seems to be due to the particles being forced so closely together as to bring molecular forces into action. The office of the water seems to be to ex-

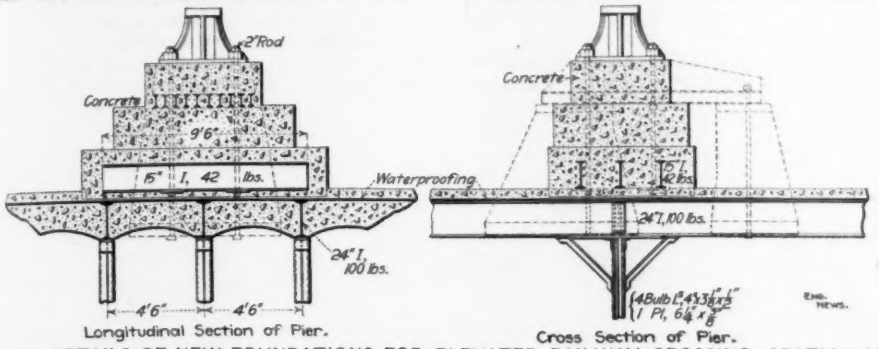


FIG. 68. DETAILS OF NEW FOUNDATIONS FOR ELEVATED RAILWAY CROSSING, SECTION V-b, AT 53D ST.

pel the air and thus permit the faces of the small particles to come into intimate contact. The adhesion between the minute particles is very much the same as that of two sheets of glass that have been forced into intimate contact by sliding one over the other—with or without a film of water intervening. In consequence of this molecular action, some stones when broken into small fragments, wetted and traversed by heavy wheels or by a road roller, will be cemented together to a considerable degree. This cementation is due to the fact that the friction of one small piece of stone upon another produces a very fine powder at a point of contact, which, when wetted and compressed, forms a weak cement. Owing to the round surfaces of water-worn pebbles, this cementing action is much less with gravel than with rough angular fragments of broken stone; but with gravel composed of undecayed rocky fragments this action takes place to a considerable degree. As a rule, pebbles of a bluish color will thus cement together, while reddish or brown ones will not, which accounts in part at least for the well-known superiority of blue gravel for road purposes. Trap rock possesses the property of cementation in a high degree, and hence trap gravel is a very excellent road building material. Limestone possesses a fair degree of cementation, but is too soft to wear well. Quartz wears well, but produces little or no dust for cementation, and besides its surfaces are so smooth and hard that the binder has but little effect; and therefore it rarely happens that a gravel of which more than one-half of its bulk is white quartz pebbles proves to be a good road gravel.

STUDY OF ROAD GRAVELS.—To test the above theory, the writer has made a study of several well-known road-building gravels. The samples were in each case selected by a competent man who was thoroughly acquainted with the practical use of the particular gravel, and who took pains to secure a representative sample. The source of the several samples is stated in Table I.

Each sample was first tested to determine the amount of native binding material present. The gravel was washed until the water was perfectly clear. The wash-water was then allowed to stand until all matter in suspension had settled. The precipitate was called clay, although a small part of it may have been very finely divided silica. A very small amount of soluble matter may have been carried away in the water, but the error cannot be material. The amount of clay in each example is stated in Table I.

A sieve analysis was then made of the washed gravel, with the results shown in Table I.

The per cent. of voids in the washed gravel also is shown in Table I. In passing it may be wise to call attention to the fact that the per cent. of voids in gravel used for road purposes is not as important as in the material employed in concrete, since in the former the pressure of the roller or of the wheels compacts the material more than ramming does the concrete, and possibly also crushes part of the material, and thus still further reduces the voids.

Mineralogical analyses of the gravel were made with the results shown in Table II.

What can be learned from Tables I. and II.? In answering this question the several gravels will be considered in order.

1. URBANA.—This is screened drift gravel obtained near Urbana, Champaign Co., Ill., which has been used in a few instances on private driveways. Table I. shows only 3.8% of clay present; and consequently the gravel is deficient in binder unless there is some ingredient besides clay that will cement the mass together. An examination of Table II. shows 65% of "limestone with varying amounts of iron." Most of this limestone is in rounded pebbles, only a small part of it being in flat chips that can be easily crushed under light traffic, and consequently the limestone probably furnishes only a little binding material. The per cent. of iron present is so small as to be wholly inappreciable as a binder. There is only a small per cent. of crystalline rocks present, and these are hard and not readily crushed by the wheels of vehicles and feet of horses; and consequently cannot materially affect the binding qualities of the material. The gravel contains 22.2% of quartz; but this material is very hard and not easily crushed, and besides the dust is almost wholly devoid of cementing properties. Both the quartz and the crystalline rocks are quite sharp and angular, which is a very desirable condition. The last line of Table II. shows the per cent. of iron in the clay. This iron is probably immediately available for binding material, and certainly acts mechanically to bind the coarser particles together and probably also acts chemically as a true cement; but the proportion of iron is too small in comparison to the entire mass of gravel to have an appreciable effect.

All the evidence of Table II. tends to show that this gravel will not make a good road, particularly under light traffic; and experience in actual service agrees with this conclusion. The gravel did not pack well, and the clay and finer material worked to the bottom, leaving a surface coat of loose and comparatively large pebbles.

TABLE II.—Mineralogical Analyses of Road-Building Gravels.

Ref. No.	Mineral constituents.	1. Urbana.	2. Decatur.	3. Lexington.	4. Rockford.	5. Peekskill.	6. Buck Hill.	7. Rock Hill.	8. Shark River.
		Per cent.							
1.	Limestone, pure carbonate of lime	....	21.9	....	....	....	....	....	....
2.	" ferruginous	65.0	8.1	21.0	....	33.5	....	....	....
3.	" with iron and silica	....	....	....	73.8	....	....	....	....
4.	" siliceous	....	....	58.0	....	....	....	....	....
5.	Sandstone, ferruginous	....	7.1	....	....	8.7	....	....	....
6.	" finely divided	....	....	....	....	....	....	10.5	....
7.	" coarse	....	....	....	....	....	....	....	....
8.	Metamorphosed rock, iron, silica, mica	....	....	....	....	....	....	3.8	....
9.	" " iron, silica, limestn	....	....	....	....	4.9	....	....	....
10.	Conglomerate, iron	....	....	....	1.7	....	6.0	....	....
11.	" limestone and quartz	....	....	....	12.3	....	....	....	....
12.	Crystalline rocks, acidic	4.6	10.1	1.4	....	15.5	....	....	....
13.	" " basic	4.3	19.7	1.1	4.3	14.2	....	....	....
14.	Quartz	22.2	23.3	15.7	4.6	22.1	73.9	64.3	78.8
15.	Clay	3.8	9.6	4.2	3.2	0.9	20.6	20.7	20.2
	Total	99.9	99.8	99.4	99.9	99.8	99.9	99.3	99.0
	Iron in the clay	7.6	9.1	9.1	9.4	....	16.5	18.1	9.9

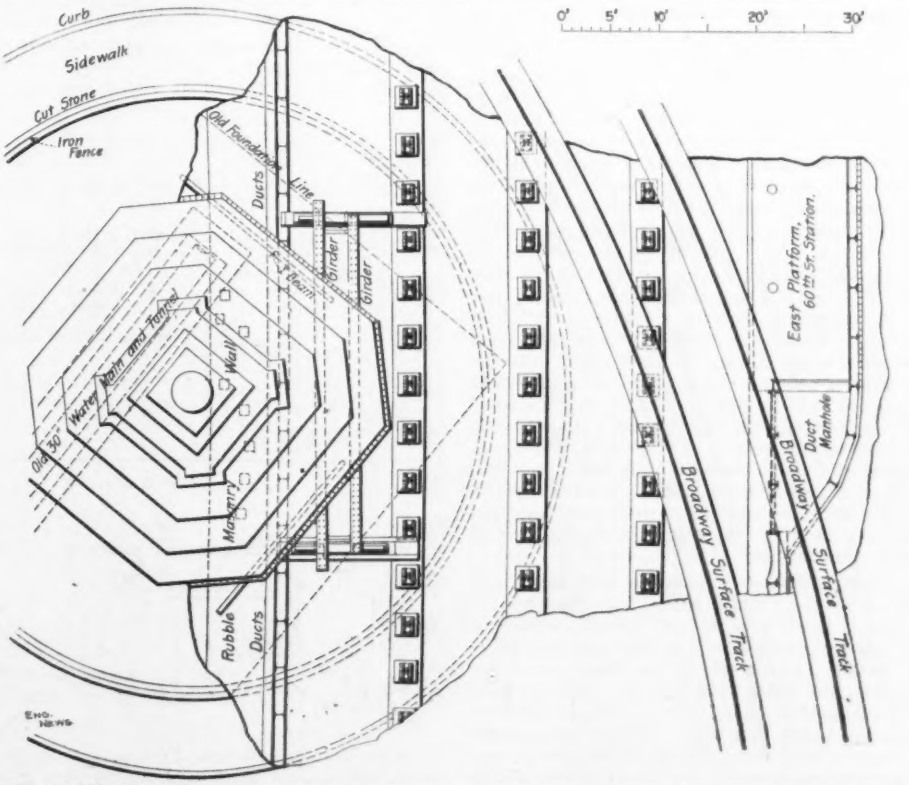


FIG. 69. PLAN SHOWING METHOD OF SUPPORTING COLUMBUS MONUMENT DURING CONSTRUCTION OF SUBWAY ON SECTION V-b.

2. DECATUR.—This is a gravel much used on country roads near Decatur, Macon County, Ill., with satisfactory results. An examination of Table II. shows that it contains more than twice

as much clay as the Urbana gravel, but only about one-third enough to fill the voids. A considerable portion of the limestone, both the pure and the ferruginous—a total of 30%—is in thin friable chips, and is easily crushed by the traffic, thus making an excellent binder. The ferruginous limestone contains an unimportant amount of iron, but the ferruginous sandstone is heavily

The writer is not familiar with the method of constructing roads with this gravel, and does not know whether or not the material will compact under the traffic in a reasonable time; but he has seen a road in process of construction in which the gravel was bound under the action of a sprinkler and a steam roller, and is informed that a 4 to 6-in. layer has been compacted with a 2

voids is no guide in judging the quality of a gravel for road purposes. Nor is the amount of clay present a sure index of the road building qualities of a gravel, for in the last three samples clay is the sole binder, while in the first five it is a comparatively unimportant element. The defects of clay as a binder were discussed in the first part of this article. The last three gravels

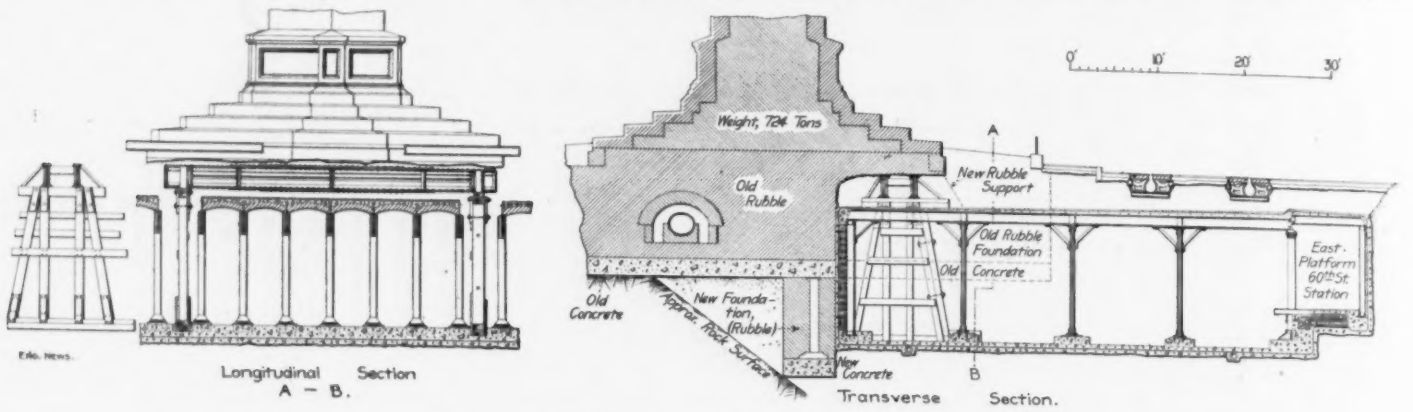


FIG. 70. SECTIONAL ELEVATIONS SHOWING METHOD OF SUPPORTING COLUMBUS MONUMENT.

charged with iron oxide, which makes a good cementing material. This gravel makes a smooth hard surface, reasonably free from dust in the summer and mud in the winter.

This gravel has a little greater proportion of iron in the clay than the Urbana gravel, and its behavior on the road seems to indicate that the iron is an important element in binding the mass together; at least if the stratum of clay rich in iron is omitted, the road ravels much more easily than if it is included. Only a very elaborate and careful scientific investigation could determine the exact nature and the relative proportions of the forces that actually bind the gravel together in the road.

3. LEXINGTON.—This gravel is used in and around Lexington, McLean County, Ill., for county highways with entire satisfaction. Notice that the clay is equal to only one-seventh of the voids. Nearly all of the 21% of ferruginous limestone consists of thin chips which are easily crushed by the traffic. Some binder must be obtained from the 58% of silicious limestone. The per cent. of crystalline rocks present is very small, and cannot very materially affect the quality of the gravel. The amount of quartz is less than in the preceding gravels, and is an unimportant element.

4. ROCKFORD.—This gravel has given satisfactory service in Rockford, Winnebago County, Ill., probably under more exacting conditions than any of the preceding. Notice that this gravel contains, roughly speaking, only about one-tenth enough clay to fill the voids. The chief source of binder is the limestone which exists in the form of pebbles, but contains no considerable amount of iron or silica. The basic crystalline rocks by decomposing may furnish a little binder; but as they are round, hard pebbles, not easily crushed, the binder derived from this source can be of no practical importance. A very little cementing material may be derived from the iron conglomerate and also from the limestone and quartz conglomerate.

5. PEEKSKILL.—This gravel is from Roa Hook, a "point" in the Hudson River, near Peekskill, N. Y., and is much used in and around New York city, and is considered one of the best road gravels. Notice that the clay is less than 1-30 of the volume of the voids. Considerable binding material is doubtless derived from the ferruginous limestone, which, however, contains a comparatively small per cent. of iron. The iron in the ferruginous sandstone is too small in amount to be appreciable. Considerable binder is derived from the metamorphosed rocks containing iron, silica, and mica. Notice that there are nearly 30% of crystalline rocks, which upon being finely pulverized will decompose and furnish an excellent cementing material. This is what is called a blue gravel.

or 3-ton horse-roller and copious sprinkling. Practically the only binder in this gravel is produced by the crushing of some of the pebbles.

6. BUCK HILL.—This gravel was obtained from the Buck Hill pit at Tuckahoe, N. J., and was recommended to the writer by Hon. Henry I. Budd, State Commissioner of Public Roads of New Jersey, as a representative road-building gravel. It consists practically of clay and partially rounded quartz pebbles. The clay is probably enough to fill the voids when the gravel has been compacted by traffic. This gravel is used for road building without rolling.

7. ROCK HILL.—This sample is substantially the same as No. 6 above, except that it has a greater per cent. of voids and contains some sandstone which crushes easily and materially reduces the voids of the gravel after it has been compacted in the road. It is stated that the best results are obtained by mixing this and the preceding gravel half and half.

8. SHARK RIVER.—This gravel was obtained

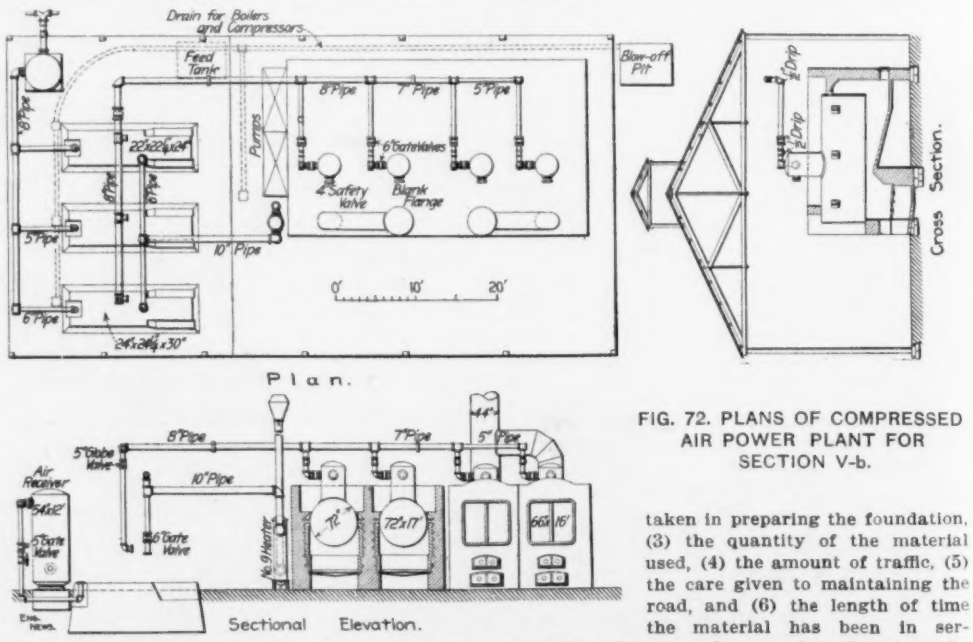


FIG. 72. PLANS OF COMPRESSED AIR POWER PLANT FOR SECTION V-b.

from the Manasquan Gravel Co., of Asbury Park, N. J., and is used much in southern New Jersey and around New York city. It consists wholly of clay and small round pebbles of pure white quartz, and consequently clay is the only binding material, except the 9.9% of iron in the clay.

COMPARISONS.—Comparing the last three gravels with the first five shows that the relation between the per cent. of clay and that of the

have more iron in the clay than the first five. The last three, particularly the Buck Hill and the Rock Hill gravels, not only have a larger per cent. of iron in the clay, but also a larger proportion of clay; and consequently the iron is proportionally more important than in the first five. Apparently the binding material in the first five is chiefly ferruginous limestone, and in the last three clay and iron.

CONCLUSION.

It is not claimed that a study as above will certainly determine the suitability of a gravel for road building; but it is claimed that such a study will throw valuable light upon the probable behavior of a gravel. The only sure way of judging the road building qualities of a gravel is an actual test on the road, for much depends upon the friability of the gravel, the weight of the traffic, the climatic conditions, etc. In applying the test of actual service, particularly to determine the relative merits of two gravels, account should be taken of (1) the nature of the soil, (2) the care taken in preparing the foundation, (3) the quantity of the material used, (4) the amount of traffic, (5) the care given to maintaining the road, and (6) the length of time the material has been in service. The character of a gravel road is generally indicated by the sound of the metal tires of the wheels of the vehicles passing over it. If the wheels make a continuous crisp, gritty sound, the road is reasonably good; if the gritty sound is absent there is probably too much earthy matter on the surface; and if the sound is intermittent or discontinuous, there are probably too many pebbles in the surface material.

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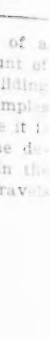

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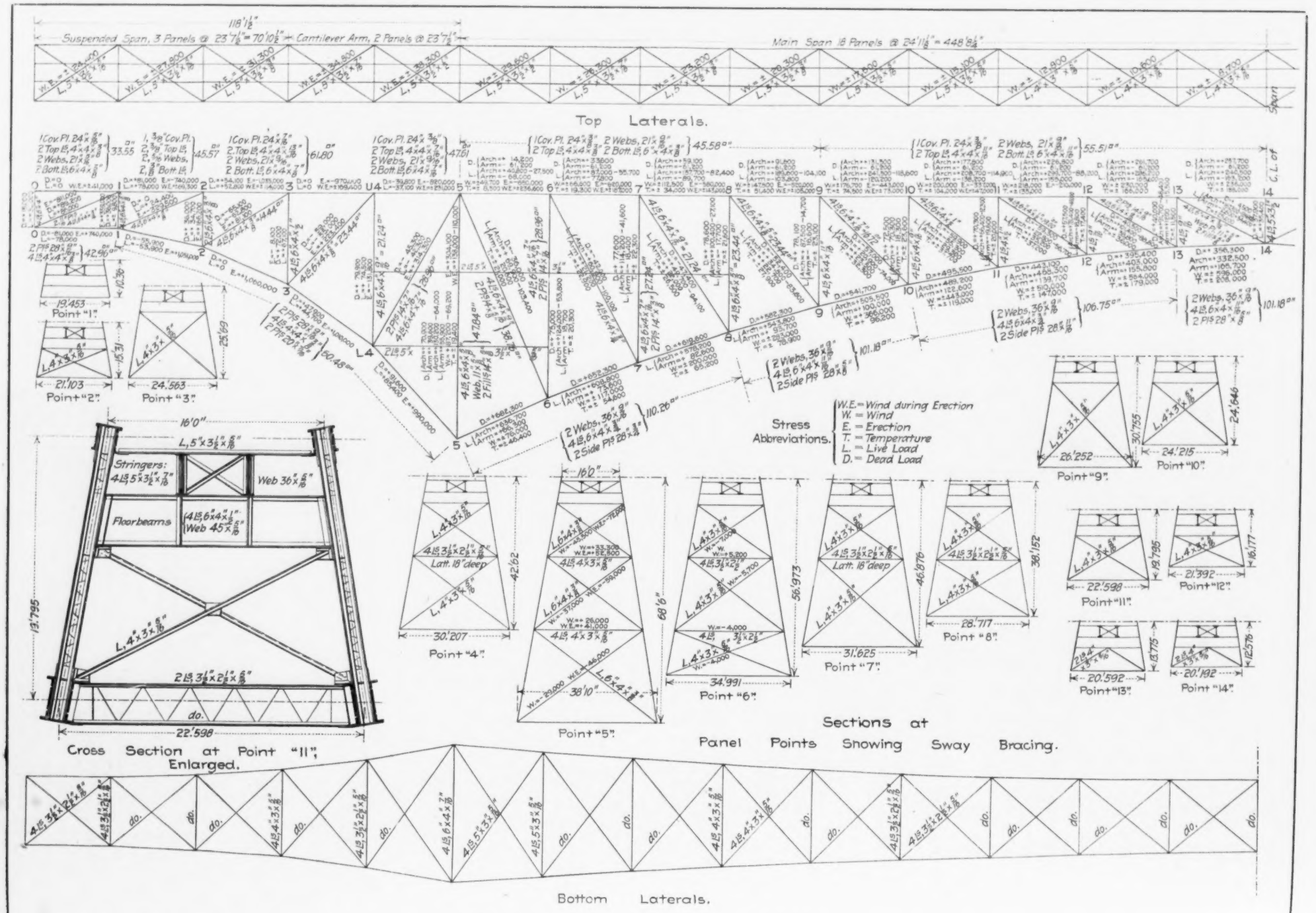
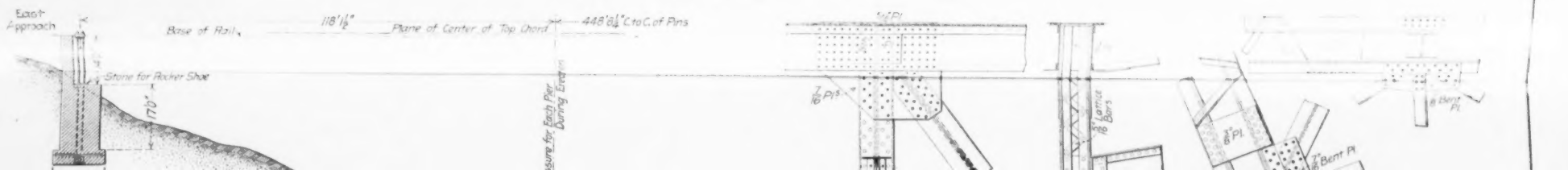


FIG. 4. STRAIN SHEET AND MAKE-UP OF MEMBERS.



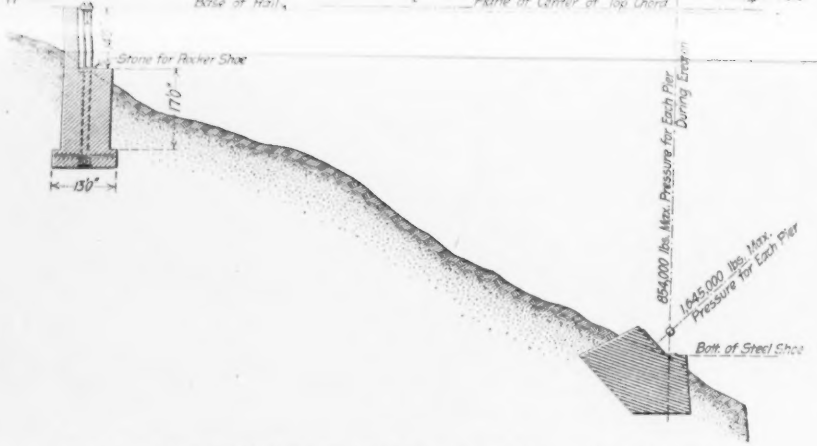

Stone for Rocker Shoe
170"
130"
854,000 lbs. Max. Pressure for Each Pier During Erection
1,645,000 lbs. Max. Pressure for Each Pier
Bott. of Steel Shoe

FIG. 2. HALF TRANSVERSE PROFILE OF GORGE, SHOWING SHORE ABUTMENT AND ARCH SKEWBACK.

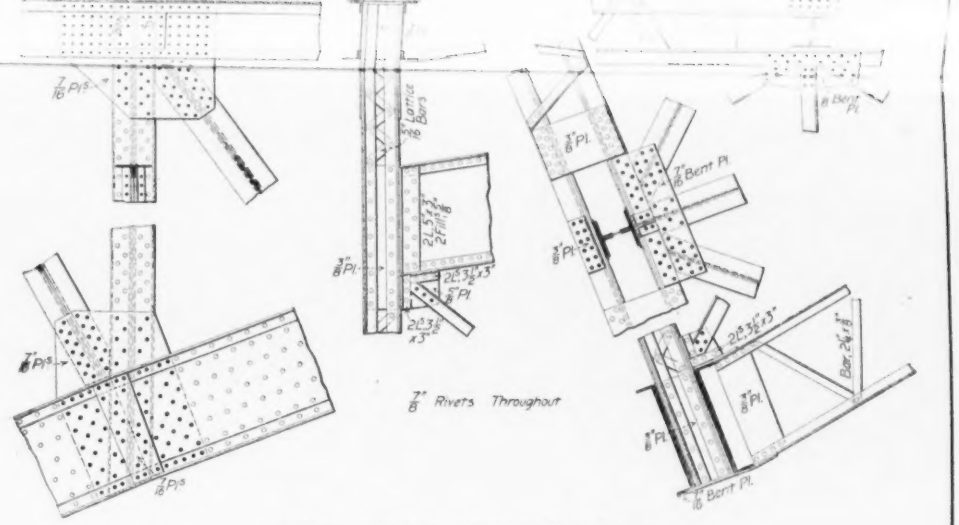


FIG. 7. TYPICAL POST DETAILS FOR MAIN ARCH SPAN.

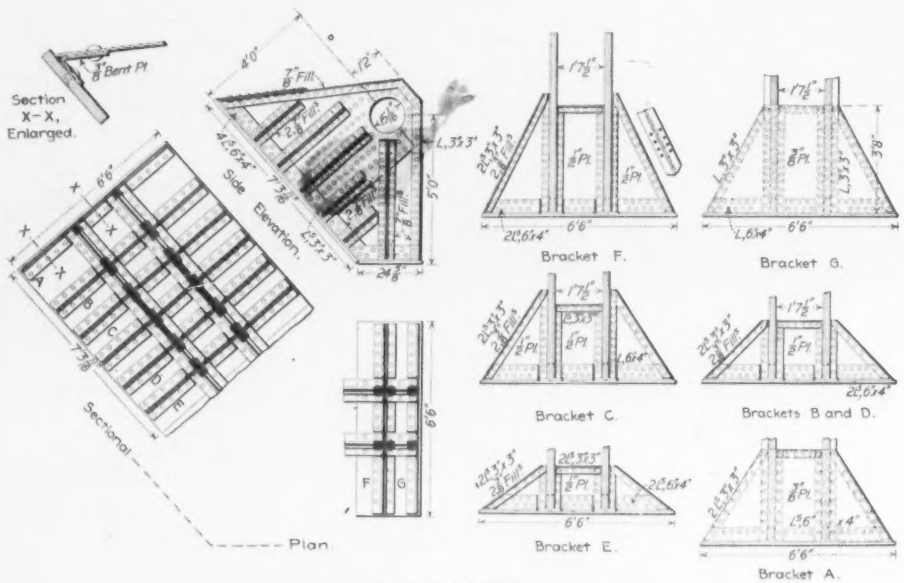


FIG. 5. DETAILS OF ROLLED STEEL ARCH SHOES.

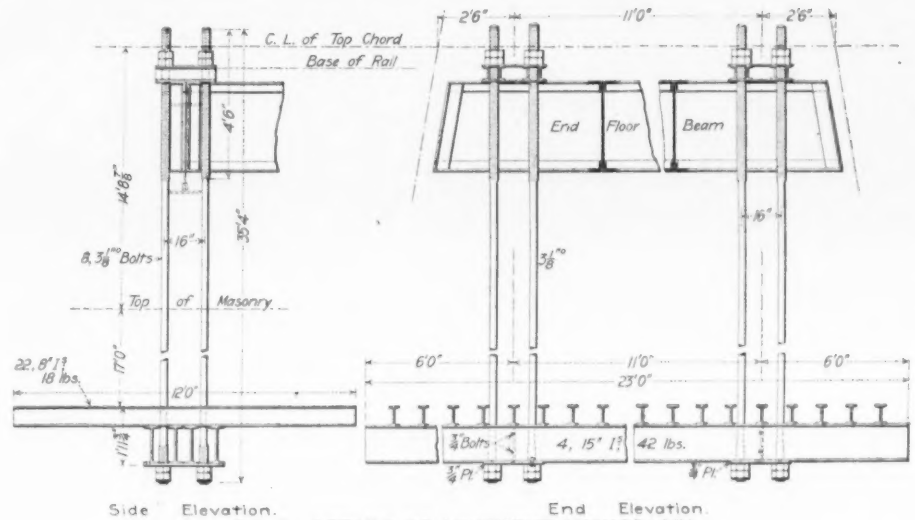


FIG. 3. DETAILS OF ABUTMENT ANCHORAGES FOR SHORE ARMS.

STEEL ARCH BRIDGE OVER THE RIO GRANDE; PACIFIC RY., COSTA RICA.

Theodore Cooper and Gunwald Aus, Mem's. Am. Soc. C. E., Engineers.

Milliken Bros., New York City, Builders.